حكومة أبوظبي
GOVERNMENT OF ABU DHABI

ROAD GEOMETRC DESIGN MANUAL

TR-514

ROAD GEOMETRIC DESIGN MANUAL

DOCUMENT TR-514

SECOND EDITION

NOVEMBER - 2021

Document No: TR-514
Second Edition
November - 2021
Department of Municipalities and Transport
PO Box 20
Abu Dhabi, United Arab Emirates

DOCUMENT AMENDMENT PAGE (TR-514)

EDITION No.	REVISION No.	DATE	CLAUSE / SECTION (PAGE)	AMENDMENTS	NOTES
01	00	DEC 2016			
02	01	NOVEMBER 2021	COVER	Modified as below: Document No: TR-514 Second Edition November - 2021 Department of Municipalities and Transport	
02	01	NOVEMBER 2021	LIST OF FIGURES (PAGE-22)	Added as follows: "Note: Some of the figures are for illustrative purposes. For detailed figures, dimensions and configurations, refer to Abu Dhabi Standard Drawings TR-541"	
02	01	NOVEMBER 2021	GLOSSARY - DEFINITIONS (PAGE-46)	Modified as below: "STEAM: Strategic Transportation Evaluation & Assessment Model of DMT"	
02	01	NOVEMBER 2021	ACRONYMS (PAGE-52)	Modified as below: DMT: Department of Municipalities and Transport DRM: Dhafra Region Municipality (Deleted DMAT, UPC and WRM)	
02	01	NOVEMBER 2021	ACRONYMS (PAGE-52)	Added as follows: RSA: Road Safety Audit NMU: Non-Motorised Users PTW: Powered two wheelers VRS: Vehicle Restraint systems ZOI: Zone of Intrusion	
02	01	NOVEMBER 2021	1.2 / PARAGRAPH #2 (PAGE-53)	Modified as below: Designers must ensure roadway designs meet operational and Road Safety requirements	
02	01	NOVEMBER 2021	TABLE 1.2 (PAGE-62)	Modified as below: Revised "DMAT" as "DMT"	
02	01	NOVEMBER 2021	TABLE 1.2 (PAGE-62)	Added as follows: "Road Safety Audit" by "ADM -Traffic Services Section"	
02	01	NOVEMBER 2021	1.4.2.15 (PAGE-68)	Modified as below: "Standard parking bay (parallel): 2.7m (min. 2.5m) x 6.0m " Also, "Note" deleted.	
02	01	NOVEMBER 2021	TABLE 1.5 (PAGE-78)	Added as follows: "Road Safety Audit"	
02	01	NOVEMBER 2021	1.5.1 (PAGE-78)	Added as follows: "Road Safety Audit	
02	01	NOVEMBER 2021	1.5.9 (PAGE-84)	Modified as below: "Safety considerations, including Road Safety Audits requirements."	
02	01	NOVEMBER 2021	1.5.24 (PAGE-93)	Added as follows: "Road Safety Audit Report"	
02	01	NOVEMBER 2021	TABLE 2.6 & 2.7 (PAGE-107 & 108)	Added as follows: "Note: As per the Executive Council resolution 1075/ECAS dated 19-07-2017, speed margin between the posted speed and the speed cameras on the main	

EDITION NO.	REVISION NO.	DATE	CLAUSE / SECTION (PAGE)	AMENDMENTS	NOTES
				roads is removed; therefore posted speeds on the main roads where speed cameras are installed shall be equal to the design speed.”	
02	01	NOVEMBER 2021	2.6.1 / PARAGRAPH #2 (PAGE-109)	Modified as below: “City-Bus M is applicable for streets with designated public bus routes and/or school bus routes at school zones”	
02	01	NOVEMBER 20 21	2.9 / LAST PARAGRAPH (PAGE-125)	Added as follows: “Generally, embankment height between 1.5m-2.0m is used and can be more than 3.0m at drainage facilities and utility locations or Sabkha areas. Embankment heights shall be studied and minimized as much as possible unless any special cases such as Sabkha areas. Utility providers’ requirements for existing/proposed networks shall be optimized. Drainage facilities shall be provided in natural stream/depressed areas in order to minimize their negative impacts on the road elevation, where applicable.”	
02	01	NOVEMBER 2021	2.12 (PAGE-127)	Modified as below: Road Safety	
02	01	NOVEMBER 2021	2.12 PARAGRAPH #11 (PAGE-129)	Added as follows: “The design team should assess their design against TR-518, the Emirate of Abu Dhabi Roadside Design Guide to ensure roadside safety is properly considered.”	
02	01	NOVEMBER 2021	2.13 (PAGE-129)	Added as follows: (new) Section 2.13 Road Safety Audit	
02	01	NOVEMBER 2021	3.2.3.1 LAST PARAGRAPH (PAGE-136)	Modified as below: “Shoulders shall not be applied on urban streets (other than urban freeways and expressways)”	
02	01	NOVEMBER 2021	3.2.3.4 BULLET #2 & 3 (PAGE-137)	Modified as below: •For freeways and expressways, 1.2m inside shoulder and 3.0m outside shoulder shall be applied. •For divided rural collector roads, 1.2m inside and outside shoulder shall be applied •For undivided rural local roads, shoulder is not applicable	
02	01	NOVEMBER 2021	3.2.4.4 (PAGE-141)	Added as follows: “It should be noted that rumble strips have minimal effect on speed reduction and are more of an alerting feature than a speed reduction feature.”	
02	01	NOVEMBER 2021	3.2.6 (PAGE-141)	Added as follows: “As highlighted in TR-518, kerbing has been shown to be a major contributor to the vaulting and destabilization of impacting errant vehicles, particularly at high speeds and with higher kerbs.”	
02	01	NOVEMBER 2021	3.2.6.2 (PAGE-143)	Modified as below: “Kerb Upstand; Height of the typical upstand kerb (Type-B) to be 100 mm for boulevards, avenues, streets, access lanes, frontage lanes and parking areas”	
02	01	NOVEMBER 2021	3.2.6.2 (PAGE-143)	Modified as below: “Kerb Type A – This 50mm upstand kerb was commonly used on Abu Dhabi streets, access lanes and	

EDITION No.	REVISION No.	DATE	CLAUSE / SECTION (PAGE)	AMENDMENTS	NOTES
				frontage lanes from 2016 till the directives issued in 2018. Details are retained for information in case of maintenance works involving existing kerbs at this height.”	
02	01	NOVEMBER 2021	3.2.6.2 (PAGE-144)	Modified as below: “Kerb Type-B1 – This 150mm high upstand kerb…”	
02	01	NOVEMBER 2021	3.2.6.2 (PAGE-144)	Modified as below: “Kerb Type D - This flush kerb is used between the road pavement and pedestrian ramps.”	
02	01	NOVEMBER 2021	3.2.6.2 (PAGE-144)	Modified as below: “Kerb Type F - This flush kerb is used between asphalt pavement and interlocking vehicular pavement”	
02	01	NOVEMBER 2021	3.2.6.2 (PAGE-144)	Added as follows: “Rolled Kerb – This 50mm high dropped roll kerb may be used as an option for (1x1) streets in Residential Context areas with low operating speeds less, low traffic volumes, where a proper edge zone is included to provide protective elements such as bollards, planting, planters and other street furniture”	
02	01	NOVEMBER 2021	3.2.6.4 (PAGE-144)	Modified as below: “•On urban boulevards, and avenues, streets, access lanes, frontage lanes and parking areas, 100 mm high upstand curb is typically used” Also 2nd and 4th bullets are deleted	
02	01	NOVEMBER 2021	FIGURE 3-9 (PAGE 147)	Added “Rolled Kerb” details	
02	01	NOVEMBER 2021	3.3.2 BULLET #10 (PAGE-149)	Added New Bullet as follows: “10. Median width must take into account the ZoI of the RRS/VRS to be used and ensure that the street lighting and foundations are not within this ZoI and at risk of being struck (also refer to TR-518).”	
02	01	NOVEMBER 2021	3.3.3.2 BULLET #8 (PAGE-154)	Added New Bullet as follows: “8. Landscaping. Where raised medians are landscaped with vegetation, due consideration needs to be given to how this will be maintained especially on 80kph roads or higher.”	
02	01	NOVEMBER 2021	3.3.3.3 BULLET #4 (PAGE-154)	Added New Bullet as follows: “4. Road Safety. The adverse road safety implications should be analyzed considering risk of errant vehicle losing control and overturning or hitting rigid objects in the depressed median”	
02	01	NOVEMBER 2021	3.5 PARAGRAPH #4 (PAGE-161)	Added as follows: “All barrier types have a zone of intrusion (ZoI) which ranges from around 0.5m for concrete barriers due to high sided vehicles leaning over during an impact, to around 1.6m for typical W-beam safety fence and over 2m for normal wire rope safety fence. Also refer TR-518”	
02	01	NOVEMBER 2021	3.5 PARAGRAPH #5 (PAGE-161&162)	Added as follows: “Systems should ideally meet the requirements for MASH (the minimum acceptable standard is NCHRP-350) and it should be noted that systems worldwide are generally not tested at 140kph or 160kph speeds and therefore may not work as intended.”	

EDITION No.	REVISION No.	DATE	CLAUSE / SECTION (PAGE)	AMENDMENTS	NOTES
02	01	NOVEMBER 2021	3.7.4.1 LAST BULLET (PAGE-168)	Added last bullet as follows: "•Wire Rope Safety fence is the preferred RRS in areas subject to blowing sand."	
02	01	NOVEMBER 2021	FIGURE 3.24 (PAGE-180)	Modified vertical clearance as below: "5.5 Min."	
02	01	NOVEMBER 2021	3.12 (PAGE-181)	Added as follows: 1) Item No "3.12" 2) Add "Cul-De-Sacs and Turnarounds should be designed in a way that discourages vehicles from mounting the kerb and using the Non-Motorised users (NMU) path to access nearby through roads" at the end of the paragraph	
02	01	NOVEMBER 2021	6.2.3 (PAGE-264)	Modified as below: "1. Uncurbed Roadways. It is acceptable to provide a minimum longitudinal gradient of approximately 0.0% without affecting the requirements in Clause 6.4.1.5, but only if the roadway will not be curbed in the future…….." "2. Curbed Streets: The median edge or centerline profile on roadways and streets with curb and gutter desirably should have a longitudinal gradient of at least 0.2% for curbed local roads and collectors, 0.3% for curbed arterials, freeways and expressways."	
02	01	NOVEMBER 2021	TABLE 7-3 (PAGE-351)	Modified as below: "Left shoulder width= 1.2m" Also Note 1a and 1b deleted.	
02	01	NOVEMBER 2021	TABLE 8-1 (PAGE-366)	Modified as below: "Shoulder width= 1.2m"	
02	01	NOVEMBER 2021	TABLE 8-2 (PAGE-367)	Modified as below: "Shoulder width= Not Applicable" "Note-2: A 0.6m shy should desirably be inserted when a barrier is present."	
02	01	NOVEMBER 2021	8.2.3.1 PARAGRAPH #2 (PAGE-370)	Modified as below: "Refer to Table 8-1 and 8-2 for shoulder widths"	
02	01	NOVEMBER 2021	8.2.3.1 PARAGRAPH #2 (PAGE-370)	Modified as below: "Where cyclists are to be accommodated on the shoulder, the designer should provide a minimum paved width of 1.5 m"	
02	01	NOVEMBER 2021	8.2.4 PARAGRAPH #2 (PAGE-371)	Modified as below: "Desirably, the full width of the roadway including shoulders, footpath and cycle track on the approach roadways shall continue on the bridges."	
02	01	NOVEMBER 2021	TABLE 8-9 & 8-10 (PAGE-378 & 379)	Modified as below: "Left shoulder width= 1.2m" Also Note 2b deleted.	
02	01	NOVEMBER 2021	8.3.2.2 (PAGE-382)	Modified as below: "Where cyclists are to be accommodated on the shoulder, the designer should provide a minimum paved width of 1.5 m" Added as follows: "The cyclists are not allowed to use freeways, expressways and highways"	

EDITION NO.	REVISION NO.	DATE	CLAUSE / SECTION (PAGE)	AMENDMENTS	NOTES
02	01	NOVEMBER 2021	9.3.6.2 (PAGE-395)	Modified as below: "On urban boulevards, avenues, streets, access lanes, frontage lanes and parking areas, 100 mm high upstand curb is typically used"	
02	01	NOVEMBER 2021	9.3.6.3 BULLET #4 (PAGE-400)	Modified as below: "4. Angled Parking: For 1-way local roads, access roads and parking areas, diagonal or parallel parking is preferred whereas perpendicular parking may also be applied in special cases, such as traffic circulation and parking demand; however, road safety issues shall also be taken into consideration. For 2-way local roads, access roads and parking areas perpendicular and parallel parking shall be applied"	
02	01	NOVEMBER 2021	9.3.6.3 BULLET #7 (PAGE-401)	Added as follows: "Prohibit parking within 10.0 m of the beginning of the curb radius on the leg to any intersection with a Yield control, flashing beacon, stop sign. In addition prohibit parking within the approach and exit to a Traffic Signal due to visibility to Pedestrians and Primary Signal Heads and maneuvering difficulties"	
02	01	NOVEMBER 2021	9.3.6.3 BULLET #7 (PAGE-402)	Added as follows (last bullet): "Where Parking demand is high, parallel parking may be allowed as long as it does not affect visibility on both approach and exit"	
02	01	NOVEMBER 2021	9.3.6.4 BULLET #3C (PAGE-402)	Modified as below: "Pedestrian & Cycle Refuge. A minimum width of 2.0 m is required for a pedestrian refuge. Where there is heavy pedestrian activity or a cycle route, provide a minimum median width of 3.0 m."	
02	01	NOVEMBER 2021	9.3.6.7 (PAGE-404)	Added as follows: "All Cycle Facilities designs or schemes must be subject to Road Safety Audits Stage- 1 (at Preliminary Design) and Stage-2 (at Detailed Design) followed by Stage-3 (Pre-opening) in accordance with TR-540; the focus should be on NMU connectivity to the facility, positive protection/separation from roadway traffic and speed calming features along the cycle route."	
02	01	NOVEMBER 2021	9.3.6.9 BULLET #5 (PAGE-404)	Modified as below: "5. Cyclists. Frontage lanes may include a shared lane for cycles in the same direction of traffic. Additional cycle track may be needed for the opposite direction for cyclists. Cyclists should not be riding against vehicular traffic on the frontage lanes"	
02	01	NOVEMBER 2021	9.3.6.9 BULLET #10 (PAGE-405)	Added as follows: "Side median should provide adequate buffer and lateral clearance for street furniture on both sides of the travelled lanes."	
02	01	NOVEMBER 2021	9.3.8 (PAGE-406)	Added last paragraph as follows: "Ensure all structural elements have adequate ZOI (Zone of Intrusion) between them and any required road restraint system; refer to TR-518."	
02	01	NOVEMBER 2021	TABLE 9-10 TO 9-19	Modified as below: "Parking Width: 2.7m"	

<table>
<tr><th>EDITION No.</th><th>REVISION No.</th><th>DATE</th><th>CLAUSE / SECTION (PAGE)</th><th>AMENDMENTS</th><th>NOTES</th></tr>
<tr><td></td><td></td><td></td><td>(PAGE-411 to 420)</td><td></td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>FIGURE 10-4 (PAGE-437)</td><td>Added as follows:
“Note: Provide physical separation between through and left turning movements for Lead-Lag junctions.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.3.1.11 LAST PARAGRAPH (PAGE-466)</td><td>Added as follows:
“In all cases consideration must be given for visibility to NMUs at the crossings at these junctions.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>FIGURE 10-26 (PAGE-475)</td><td>Modified as below:
Figure revised. DUPM Technical Circular No: DUPM/USO/UPS/IP/OUT/2018/387 dated 25-February-2018; i.e. width of slip ramp to be 4m+0.5m (shy) and exit radius to be 10m</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>FIGURE 10-26 (PAGE-475)</td><td>Note added as follows:
“If considerable volume of traffic flow occurs by buses and trucks resulting in large size vehicles being the predominant design vehicles using the right turn slip lane, the width and entry/exit radii of the slip lane maybe increased as per the swept path analysis.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.4.3 PARAGRAPH #1 (PAGE-476)</td><td>Modified as below:
“For urban slip lanes, the curb-to-curb width will typically be 4.5 m. Do not use widths greater than 4.5 m as they may encourage drivers to try to use the slip lane as two lanes.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.4.7 BULLET #5 (PAGE-484)</td><td>Modified as below:
“Design acceleration lanes at intersections in the same manner as for interchange ramps using the parallel design. See Section 12.6.2. However, those distances for interchanges may be excessive under restricted urban conditions, where the length of acceleration lanes for urban intersections may be suitably adjusted. Taper lengths may vary from 30 m to 45 m instead of 90 m. The below values are recommended where acceleration lanes are required at Signalized Intersections and where right-in right-out (RIRO) located on urban expressways and arterial roads. Traffic studies and Road Safety audit shall be undertaken for selecting the proper acceleration length as per the site constraints.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.4.7 BULLET #5 (PAGE-484)</td><td>New Table Added as follows:
<table>
<tr><th>Design Speed (kph)</th><th>Painted Nose for RIRO’s Length (m)</th><th>Acceleration Lane Length (m) (Excluding Taper & Nose)</th><th>Taper Length (m)</th></tr>
<tr><td>80</td><td>-</td><td>40 to 90</td><td>30</td></tr>
<tr><td>100</td><td>20</td><td>80 to 110</td><td>40</td></tr>
<tr><td>120</td><td>40</td><td>100 to 130</td><td>45</td></tr>
</table></td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.6.1 BULLET #3 (PAGE-490)</td><td>Added as follows:
“The refuge Island should be adequately sized for pedestrian and cyclists storage.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.6.2.1 (PAGE-491)</td><td>Added last bullet as follows:</td><td></td></tr>
</table>

<table>
<tr><th>EDITION No.</th><th>REVISION No.</th><th>DATE</th><th>CLAUSE / SECTION (PAGE)</th><th>AMENDMENTS</th><th>NOTES</th></tr>
<tr><td></td><td></td><td></td><td></td><td>“the use of flush islands for separation of opposing traffic streams should be avoided”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.6.3.1
BULLET #4
(PAGE-492)</td><td>Modified as below:
“The minimum width for a flush-type island on new construction is 3.0 m and for reconstruction projects 1.5 m”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.7.2.2
PARAGRAPH #1
(PAGE-504)
FIGURE 10-38
(PAGE-505)</td><td>Modified as below:
“…………. For urban areas, the functional length will be the taper length plus the storage length or the deceleration length plus the taper length, whichever is larger. “</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.7.2.2
BULLET #2C
(PAGE-507)</td><td>Modified as below:
“Distances shown in Section 12.6.1 may be applied on urban facilities; however, this is not always feasible due to restricted urban conditions. Length of deceleration lanes for urban intersections may be suitably adjusted as per the design speed and/or traffic studies justifying additional storage lane requirement. Taper length to be 30 m instead of 75 m. The below values are recommended where deceleration/storage lanes are required at Signalized Intersections and where right-in right-out located on urban expressways and arterial roads.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>10.7.2.2
BULLET #2C
(PAGE-507)</td><td>New Table Added as follows:
<table>
<tr><th>Design Speed (kph)</th><th>Painted Nose for RIRO’s Length (m)</th><th>Deceleration Lane Length (m) (Excluding Taper & Nose)</th><th>Taper Length (m)</th></tr>
<tr><td>60</td><td>-</td><td>25 to 40</td><td>30</td></tr>
<tr><td>80</td><td>-</td><td>55 to 80</td><td>30</td></tr>
<tr><td>100</td><td>20</td><td>80 to 110</td><td>30</td></tr>
<tr><td>120</td><td>40</td><td>100 to 120</td><td>30</td></tr>
</table></td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>TABLE 11-4
(PAGE-547)</td><td>Modified “Safety->Disadvantages-> Bullet #1”as below:
“Increase in low-severity single-vehicle and fixed-object crashes compared to other intersection treatments.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>TABLE 11-4
(PAGE-547)</td><td>Added Bullet #3 to “Safety->Advantages” as follows:
“•Lower rates of fatal and serious crashes”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>11.7.5
PARAGRAPH #3
(PAGE-559)</td><td>Added at the end of the paragraph as follows:
“The offside kerb line projection should guide drivers around the central Island.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>TABLE 11-7
(PAGE-561)</td><td>Added Note #3 under the table as follows:
“*** Multilane Roundabouts: From local experience 85m ICD appears to be one of the safest designs when dealing with 3 lane roundabouts especially at higher speeds.”</td><td></td></tr>
<tr><td>02</td><td>01</td><td>NOVEMBER 2021</td><td>11.7.8.3
PARAGRAPH #3
(PAGE-567)</td><td>Modified as below:
“Multilane circulatory roadway lane widths typically range from 4.0 m to 4.9 m. Use of these values results in a total circulating width of 8.0 m to 9.8 m for a two-lane circulatory roadway and 12.0 m to 14.6 m total width for a three-lane circulatory roadway.”</td><td></td></tr>
</table>

EDITION No.	REVISION No.	DATE	CLAUSE / SECTION (PAGE)	AMENDMENTS	NOTES
02	01	NOVEMBER 2021	11.8.4 NEW CLAUSE (PAGE-573)	New Clause Added as follows: “Roundabout Visibility Checks”: Checking visibility for drivers approaching, driving through or exiting roundabouts is very important safety aspect to be considered when designing roundabouts. Engineers shall consider these checks to ensure different visibility envelopes are clear from any obstruction to line of sight. The following visibility checks to be carried out by designers • Forward Visibility on approach • Forward visibility at entry • Visibility to the left • Circulatory Visibility • Pedestrian crossing visibility For further details, refer to British Standards for Highways DMRB (Design Manual for Roads and Bridges) - Geometric Design of Roundabouts	
02	01	NOVEMBER 2021	12.3.7 BULLET #1 (PAGE-584)	Modified as below: “Figure 12-8”	
02	01	NOVEMBER 2021	12.5.4 BULLET #1 (PAGE-646)	Modified as below: •Where the truck ADT is greater than 15%, the design speed for the initial curve after the exit curve shall be determined to facilitate the safety movement of the traffic. •Use a design speed of 50 km/h (maximum) for cloverleaf interchange loop ramps between freeways.	
02	01	NOVEMBER 2021	12.5.6 BULLET #1 & TABLE 12-7 (PAGE-647 & 649)	Modified as below: “For diagonal ramps and loop ramps, the ramp travelled way width is 5.0 m with a 1.2 m wide left shoulder, and a 3.0 m wide right shoulder with no curb clearance on both sides.” Also Figure 12-50 and Table 12-7 revised accordingly.	
02	01	NOVEMBER 2021	13.2.2.6 BULLET #5 (PAGE-686)	Bullet #5 deleted	
02	01	NOVEMBER 2021	13.2.2.6 BULLET #6 (PAGE-686)	Modified as below: “Vertical separation of pedestrian paths from vehicle travel ways should be maintained. Typical kerb height for Boulevards, Avenues, Access Lanes and Frontage Lanes to be 10cm.”	
02	01	NOVEMBER 2021	TABLE 13-2 (PAGE-687)	Modified as below: Max. width of cycle track is revised as 3.0m	
02	01	NOVEMBER 2021	TABLE 13-2 (PAGE-687)	Added a Note for “Cycle Track” as follows: (2) Desirable width of 1-way cycle track is 1.5m and the absolute minimum width of 1-way cycle track is 1.2m (for retrofitting projects only). desirable width of 2-way cycle track is 3.0m and the absolute minimum width of 2-way cycle track is 2m (for retrofitting projects only)””	
02	01	NOVEMBER 2021	13.2.3.4 BULLET #6 (PAGE-690)	Modified as below: “Place the vehicle stop bar 3m ahead of the crosswalk”	

EDITION No.	REVISION No.	DATE	CLAUSE / SECTION (PAGE)	AMENDMENTS	NOTES
02	01	NOVEMBER 2021	13.2.3.6 BULLET #12 (PAGE-697)	Modified as below: “Shall provide a minimum of 4% (Refer to Table 16-1) reserved parking facilities for disabled access with minimum dimensions of (2.5+1.5) m x 5.5 m “	
02	01	NOVEMBER 2021	13.2.3.7 BULLET #1 (PAGE-697)	Modified as below: “Typical preferred curb height is 100 mm for Boulevards, Avenues, Access Lanes and Frontage Lanes”	
02	01	NOVEMBER 2021	13.3.4.2 LAST PARAGRAPH (PAGE-706)	Modified as below: “Where this is not possible and the distance between the edge of the shoulder and the shared use path is within clear zone , a suitable positive separation by physical barrier or Vehicle Restraint systems (VRS) should be provided.”	
02	01	NOVEMBER 2021	15.1 (PAGE-721)	Added last paragraph as follows: “All ITS structure, systems and sub systems infrastructure that are along the road side and within clear zone need a suitable physical barrier or Vehicle Restraint systems (VRS).”	
02	01	NOVEMBER 2021	16.3.1.2 LAST BULLET (PAGE-739)	Added last bullet as follows: “Consideration must be given to how maintenance workers and vehicles will access the landscaping and where will vehicles stop to unload mowers or pick up waste plant material like palm fronds etc.”	
02	01	NOVEMBER 2021	16.8 BULLET #1 (PAGE-760)	Modified as below: “In rural areas, a truck lay-by should be provided approximately every 25 km (maximum).”	
02	01	NOVEMBER 2021	16.8 (PAGE-760)	Added Bullet #8 as follows: “Lay-bys must be physically separated from Main line traffic by a VRS/Guard Rail with proper end termination.”	
02	01	NOVEMBER 2021	16.9.2.4 BULLET #2 (PAGE-766)	Modified as below: “Parking lots adjacent to each other (served from different aisles) should be separated by a raised sidewalk at least 1.0m wide which shall be unobstructed and clear from any signs or street lighting poles.”	
02	01	NOVEMBER 2021	16.9.2.5 BULLET #1 (PAGE-766)	Modified as below: “Typical stall widths (measured perpendicular to the parked vehicle) are 2.7 m”	
02	01	NOVEMBER 2021	FIGURE 16-15 (PAGE-768)	Modified as below: Width of parallel parking is revised as 2.7m	

TABLE OF CONTENTS

LIST OF FIGURES

Note: Some of the figures are for illustrative purposes. For detailed figures, dimensions and configurations, refer to Abu Dhabi Standard Drawings TR-541

LIST OF TABLES

GLOSSARY

Definitions

Access: Permission, liberty, or ability to enter, approach, or to make use of.

Accessibility: 1) A measure of mobility. 2) Total travel time between areas weighted by the relative attractiveness of the destination. 3) A measure of the ability of public transportation users to access transit modes.

The ability for all people, including people with impaired mobility and all ages, to physically reach desired destinations, services, and/or activities. See also Universal Design

Accessible: A site, building, facility, or portion thereof that can be approached, entered, and used by the physically impaired.

Access Lane: A very low vehicle capacity 1+1 street (one lane in each direction). Anticipated very low traffic volumes, and very low speeds.

Access Point: An intersection, driveway, or opening providing access to a roadway.

Aesthetics: The science or philosophy concerned with quality of visual experience, pertaining primarily to desirable visual values and with judgments concerning appealing design.

Alignment: 1) The fixing of points on the ground in correct linear form for setting out a road, railway, wall, transmission line, canal, etc. 2) A ground plan showing a route (as opposed to a profile section) including levels and elevations.

Alley: A narrow right-of-way to provide access to the side or rear of individual land parcels.

Angle of Turn: A measure of change in direction of vehicle.

Annual Average Daily Traffic (AADT): The total annual volume of traffic passing a point or segment of a road in both directions divided by the number of days in a year.

Annual Vehicle Kilometres Travelled: Average annual traffic on a road segment, expressed as AADT, multiplied by the number of days in the year, multiplied by the length of the road segment.

Approach: 1) A set of lanes accommodating all left-turn, through and right-turn movements arriving at an intersection from a given direction. 2) Section of the accessible route that flanks the landing of a curb ramp. The approach may be slightly graded if the landing level is below the elevation of the adjoining sidewalk.

Appurtenances: Curbs, parapets, railings, barriers, dividers, and sign and lighting posts.

Area of Concern: An object or roadside condition that may warrant safety treatment.

Arterial: Functionally classified road that is characterised by a high degree of continuity and a capacity to quickly move relatively large volumes of traffic, but often provide limited access to abutting properties. It provides for high travel speeds and the longest trip movements.

Articulated Bus: A bus that has the rear portion flexible but permanently connected to the forward portion with no interior barrier to movement between the two parts; the passenger capacity is about 60 to 80 persons seated, with room for many standees, and length is from 18 to 21 metres.

Asset Management: 1) A systematic process of operating, maintaining, and upgrading transportation assets cost-effectively by combining engineering practices and analysis with sound business practice and economic theory. 2) The management of the physical infrastructure such as pavements, bridges, and airports, as well as human resources (personnel and knowledge), equipment and materials, and other items of value (e.g. financial capabilities, right-of-way, data, computer systems, methods, technologies and partners). 3) The management of the interference with through traffic caused by traffic entering, leaving, and crossing streets.

At-Grade Intersection: An intersection where all roadways join or cross at the same level.

Auxiliary Lane: The portion of the roadway adjoining the through travelled way for parking, speed change, turning, storage for turning, weaving, truck climbing, or for other purposes supplementary to through traffic movement.

Avenue: A medium vehicle capacity 2+2 street (two lanes in each direction). Avenues may have frontage lanes.

Average Daily Traffic (ADT): The total volume during a given time period, in whole days, greater than one day and less than one year, divided by the number of days in the time period.

Average Running Speed: The sum of the distances travelled by vehicles on a road section during a specific time period divided by the sum of their running times.

Average Spot Speed: The arithmetic mean of the speeds of all traffic, or component thereof, at a specified point.

Average Travel Speed: For all traffic, or a component thereof, the summation of distances divided by the summation of overall travel times.

Axle: A supporting shaft or member on, or with which, a wheel or set of wheels revolves. The number of vehicle axles is commonly counted in pairs sharing the same axis. For example, a car is classified as having only two axles.

Axle Spacing: For each axle, the horizontal distance between the centre of that axle and that of the preceding axle; the axle spacing for the vehicle's front axle is assumed to be zero.

Axle Unit: A single axle or tandem axle.

Back Slope: The side slope in a cut section created by connecting the back of the ditch hinge point, upward and outward to the natural ground line.

Barricade: A device that provides a visual indicator of a hazardous location or the desired path a motorist should take. It is not intended to contain or redirect an errant vehicle.

Base Course: The layer or layers of specified or selected material of designed thickness placed on a subbase or sub grade to support a surface course.

Benefit-Cost Ratio: Comparison of the benefits generated by the project to the costs incurred in the project over the period of analysis.

Borrow: Suitable material from sources outside the roadway prism, used primarily for embankments.

Boulevard: A high vehicle capacity 3+3 street (three lanes in each direction). Boulevards may have frontage lanes. Existing streets with 3+3 or more lanes are also classified as boulevards.

Brake Check Area: A turnout or pull-off area designed to provide an opportunity for a driver to inspect equipment on the vehicle and to ensure that brakes are not overheated at the beginning of a descent.

Breakaway: A design feature that allows a device such as a sign, luminaire, or traffic-signal support to yield or separate upon impact. The release mechanism may be a slip plane, plastic hinges, fracture elements, or a combination of these.

Bridge: A structure spanning and providing passage over a river, chasm, road, or railroad.

Bridge Approach Railing: A roadside guardrail system preceding the structure and attached to the bridge rail system that is intended to prevent a vehicle from impacting the end of the bridge railing or parapet.

Bridge Opening: The total cross-section area beneath a bridge superstructure that is available for the conveyance of water.

Bridge Railing: A longitudinal barrier whose primary function is to prevent an errant vehicle from going over the side of the bridge structure.

Bridge Roadway Width: The clear width of a structure measured at right angles to the centre of the roadway between the bottom of curbs or, if curbs are not used, between the inner faces of the parapet or railing.

Buffer Strip: An area providing a degree of protection from certain road or transportation effects for adjacent private property or protected natural resources.

Bus: A self-propelled rubber-tired vehicle designed to carry a substantial number of passengers, commonly operated on urban streets.

Bus Lane: A lane of roadway intended primarily for use by buses, either all day or during specified periods.

Bus Shelter: Usually located at high loading points, shelters may provide seating and protection from the weather for patrons.

Bus Stop: A waiting, boarding, and lighting area usually designated by distinctive signs and by curbs or pavement markings.

Bus Turnout: 1) An area designated for a bus to pull out of the traffic lane for passengers unloading and loading; 2) A bus berthing area in a facility such as a transit centre or rail station.

Bypass: An arterial road that permits traffic to avoid part or all of an urban area.

Capacity: The maximum rate of flow at which vehicles reasonably can be expected to traverse a point on a lane or road during a specified period under prevailing traffic, roadway and signalization conditions, usually expressed as vehicles per hour; most often considered the maximum amount of traffic that can be accommodated by a roadway during peak hours of demand.

Carriageway or Roadway: Refers to the configuration of a road. A single-carriageway involves a single road that carries either a single direction of traffic or two directions of traffic, with each direction separated by specific longitudinal markings in the middle section of the roadway. A single–carriageway may have one or more lanes of traffic in each direction, and may contain specific left-turn lanes at intersections. A dual carriageway involves two sets of paved roadways, each serving traffic travelling in the opposite direction of the other, separated by a physical barrier (e.g. a median or concrete barrier).

Cast-in-Place Concrete: Concrete placed in its final location in the structure while still in a plastic state.

Catch Basin: A structure, sometimes with a sump, for inletting drainage from such places as a gutter or median and discharging the water through a conduit.

Centreline: A line indicating the division of the roadway between traffic moving in opposite directions.

Centreline Turning Radius (CTR): The turning radius of the centreline of the front axle of a vehicle.

Central Island: The area of the roundabout surrounded by the circulating roadway.

Channelization: The separation or regulation of conflicting traffic movements into definite paths of travel by traffic islands or pavement markings to facilitate the orderly movements of both vehicles and pedestrians.

Channelized Intersection: An at-grade intersection in which traffic is directed into definite paths by islands.

Clearance: 1) An unobstructed horizontal or vertical space. 2) Lateral distance from the edge of the travelled way to a roadside object or feature.

Clearing: Removal of vegetation, structures, or other objects as an item of road or transportation facility construction.

Clear Runout Area: The area at the toe of a nonrecoverable slope available for safe use by an errant vehicle.

Clear Span: The face-to-face distance between supporting components.

Clear Zone: The roadside border area, starting at the edge of the travelled way, available for safe use by errant vehicles. This area may consist of a shoulder, recoverable slope, nonrecoverable slope, clear runout area, or combination thereof.

Cloverleaf Interchange: A four-leg interchange that employs loop ramps to accommodate left turns. A full cloverleaf has ramps for two turning movements in four quadrants; all other cloverleafs are referred to as partial cloverleafs.

Collector: An intermediate functional classified road that is characterised by a roughly even distribution of their access and mobility functions. Collectors connect small towns and local roads to arterial and, in urban areas, provide land access and traffic circulation within residential, commercial, and business areas.

Commercial Vehicle: A vehicle with heavy-duty chassis and suspension designed for commercial freight or passenger haulage.

Commuter: A person who travels back and forth regularly between two points; often used in reference to a suburban resident who travels daily into the city to work. The term reverse commuting, on the other hand, is used to refer to someone who lives in the city, but travels to a job in the suburbs.

Concrete Barrier: A barrier system of reinforced concrete having a traffic face that usually, but not always, adopts some form of a safety shape.

Concurrent Flow HOV Lane: An HOV lane that is operated in the same direction as the adjacent mixed flow lanes, separated from the adjacent general-purpose freeway lanes by a standard lane stripe, painted buffer, or barrier.

Construction: The supervising, inspecting, actual building, and incurrence of all costs incidental to the construction or reconstruction of a road, including locating, surveying, and mapping. The term includes resurfacing, restoration, and rehabilitation, acquisition of rights-of-way, relocation assistance, elimination of hazards of railway grade crossing, elimination of roadside obstacles, and improvements that directly facilitate and control traffic flow (e.g. grade separation of intersections, widening of lanes, channelization of traffic, traffic control systems, passenger loading and unloading areas).

Context Sensitive Solutions (CSS): A collaborative, interdisciplinary approach that involves all stakeholders to develop a transportation facility that fits its physical setting and preserves scenic, aesthetic, historic, and environmental resources, while maintaining safety and mobility. CSS is an approach that considers the total context within which a transportation improvement project will exist.

Contractor: The individual, partnership, firm, corporation, or any acceptable combination thereof, or joint venture, contracting with whom an agency or owner enters into agreement for performance of prescribed work.

Contraflow Lane: A lane on which, during certain hours of the day, high-occupancy vehicles (HOVs) operate in a direction opposite to that of the normal flow of traffic.

Controlled-Access Roadway: A roadway to which abutting properties have no legal right of access except in accordance with the requirements of the public authority that has jurisdiction over that roadway.

Control of Access: The condition where the right of owners or occupants of abutting land, or other persons, to access, light, air, or view in connection with a roadway is fully or partially controlled by public authority.

Corner Clearance: The distance from an intersection of a public or private road to the nearest access connection, measured from the closest edge of the pavement of the intersecting road to the closest edge of the pavement of the connection along the travelled way.

Corner Radius: The actual radius of a curb at the corner.

Corridor: 1) A strip of land between two termini within which traffic, topography, environment, and other characteristics are evaluated for transportation purposes. Also for transmission of a utility. 2) A broad geographical band that identifies a general directional flow of traffic. It may encompass road and transit alignments.

Cost Benefit Analysis: Comparison of costs associated with a specific action and benefits derived from that action.

Cost-Effective: An item or action taken that is economical in terms of tangible benefits produced for the money spent.

Cover: The vertical extent of soil above the crown of a pipe or culvert. Depending on the context, may also be the vegetation, or vegetation debris, such as mulch, that exists on the soil surface. In some classification schemes, fallow or bare soil is taken as the minimum cover class.

Crash Cushion: A device that prevents an errant vehicle from impacting fixed objects by gradually decelerating the vehicle to a safe stop or by redirecting the vehicle away from the obstacle.

Crashworthy: 1) A feature that has been proven acceptable for use under specified conditions through either crash testing or in-service performance. 2) A system that has been successfully crash tested to a currently acceptable crash test matrix and test level or one that can be geometrically and structurally evaluated as equal to a crash-tested system.

Cross Drainage: 1) The runoff from contributing drainage areas both inside and outside the road right of way. 2) The transmission thereof from the upstream side of the roadway facility to the downstream side.

Cross Slope: 1) The transverse slope and/or superelevation described by the roadway section geometry. 2) The slope measured perpendicular to the direction of travel.

Crosswalk/Crossing: 1) A specified part of a road where pedestrians have right-of-way to cross. 2) That part of a roadway at an intersection included within the connections of the lateral lines of the sidewalks on opposite sides of the road measured from the curbs or, in the absence of curbs, from the edges of the traversable roadway, and in the absence of sidewalk on one side of the roadway, the part of a roadway included in the extension of the lateral lines of the sidewalk at right angles to the centreline. 3) Any portion of a roadway at an intersection or elsewhere distinctly indicated for pedestrian crossing by lines or other markings on the surface.

Crown: The shape of a tangent roadway cross section with a high point in the middle and a cross slope downward toward both edges.

Cul-de-sac: A local road open at one end only and with special provision for turning around.

Culvert: Any structure under the roadway with a clear opening of 6.1 m or less measured along the centre of the roadway.

Curb: A vertical separation between the motor vehicle travelled way and the pedestrian realm.

Curb Extension: A section of sidewalk extending into the roadway at an intersection or midblock crossing that reduces the crossing width for pedestrians and may help reduce traffic speeds. Also known as a bulb-out or curb bulb.

Curb/Pedestrian Ramp: A combined ramp and landing to accomplish a change in level at a curb. This element provides roadway and sidewalk access to pedestrians using wheelchairs.

Cuts: Sections of road located below natural ground elevation, thereby requiring excavation of earthen material.

Cut Slope: Slopes extending outward and upward from the shoulder hinge point to intersect the natural ground line.

Cycle: 1) A complete sequence of traffic signal indications. 2) A non-motorised, human powered wheeled vehicle. Can include two-wheel cycles, tandem cycles, three-wheel tricycles, cycles with trailers, etc.

Cycle/Bicycle: A vehicle propelled solely by human power upon which any person may ride, having two tandem wheels, except scooters and similar devices.

Cycle/Bicycle Facilities: Improvements and provisions made to accommodate or encourage bicycling, including parking and storage facilities, and shared roadways not specifically designated for cycle use.

Cycle/Bicycle Lane/Bike Lane: A portion of a roadway that has been designated by striping, signing, and pavement markings for the preferential or exclusive use of cycles.

Cycle Facility: Any portion of the travelled way or pedestrian realm specifically intended for the use of cyclists.

Cycle Length: The total time for a traffic signal to complete one cycle.

Cycle Track: A cycle way physically separated from motor vehicle lanes.

Cycleway/Bikeway: Any road, path, or way that in some manner is specifically designated for cycle travel, regardless of whether such facilities are designated for the exclusive use of cycles or are to be shared with other transportation modes.

Cyclist: A person using a non-motorised, human-powered wheeled vehicle for travel (with the exception of wheelchairs).

Dead-End Road: A local road open at one end only without special provision for turning around.

Delay: The increased travel time experienced by a person or vehicle due to circumstances that impede the desirable movement of traffic. It is measured as the time difference between actual travel time and free-flow travel time.

Density: 1) The weight of the material that occupies a certain volume of space. 2) The number of vehicles on a roadway segment averaged over space or the number of pedestrians per unit of area within a walkway or queuing areas.

Design Capacity: 1) The maximum number of vehicles that can pass over a lane or a roadway during one hour without operating conditions falling below a preselected design level.

Design Control: Physical factors and operational characteristics and properties that control or significantly influence the selection of certain geometric design criteria and dimensions.

Design Criteria: Criteria, coupled with prudent judgmental factors that are used in design.

Design Life: The expected length of time of acceptable performance under specified conditions.

Design Speed: A selected speed used to determine the various geometric design features of the roadway.

Design Vehicle: A vehicle, with representative weight, dimensions, and operating characteristics used to establish road design controls for accommodating vehicles of designated classes.

Design Volume: A volume, determined for use in design, representing traffic expected to use the road. Unless otherwise stated, it is an hourly volume.

Diagonal Curb Ramp: Curb ramp positioned at the apex of the curb radius at an intersection, bisecting the comer angle.

Diamond Interchange: A four-leg interchange with a single one-way ramp in each quadrant. All left turns are made directly onto or off the minor road.

Directional Distribution: The directional split of traffic during the peak or design hour, commonly expressed as a percentage in the peak and off-peak flow directions.

Directional Interchange: An interchange, generally having more than one road grade separation, with direct connections for the major left-turning movements.

Directional Split: The distribution of traffic flows on a two-way facility.

Disabled Person: An individual who has a physical or mental impairment that substantially limits one or more major life activities. Also referred to as Person with Disabilities.

Divided Roadway/Dual Carriageway: A roadway that has separate travelled ways, usually with a median, for traffic in opposite directions.

Driveway: An access from a public way to adjacent property.

Driveway Crossing: Extension of the through zone and any cycle track in the pedestrian realm across a driveway.

Dropped Lane: A through lane that becomes a mandatory turn lane on a conventional roadway, or a through lane that becomes a mandatory exit lane on a freeway or expressway. The end of an acceleration lane and reductions in the number of through lanes that do not involve a mandatory turn or exit are not considered dropped lanes.

Easement: A right to use or control the property of another for designated purposes.

Edge of Travelled Way: The line between the portion of the roadway used for the movement of vehicles and the shoulder (rural) or curb (urban) regardless of the direction of travel.

Edge Zone: The area between the face of curb and furnishings zone in the pedestrian realm.

Embankment: A structure of soil, soil aggregate, or broken rock between the embankment foundation (supporting ground) and the subgrade.

Emergency Escape Ramp: A ramp, away from the main traffic stream, intended to slow and stop out-of-control vehicles.

Encroachment: 1) Unauthorised use of roadway right-of-way or easements for such items as signs, fences, buildings, utilities, parking, storage, etc. 2) An intrusion into prescribed, restrictive, or limited areas of a road system, such as crossing a traffic lane or impacting a barrier system.

End treatment: The designed modification of the end of a roadside or median barrier.

Engineer: The engineer or engineering firm issuing project drawings and project specifications, or administering the work under the contract documents.

Engineering Judgment: The evaluation of available pertinent information and the application of appropriate principles, standards, guidance, and practices as contained in recognised and prevailing documents, for the purpose of deciding upon the applicability, design, operation, or installation of a transportation feature or features.

Engineering Study: The comprehensive analysis and evaluation of available pertinent information and the application of appropriate principles, provisions, and practices for the purpose of deciding upon the applicability or design of a geometric element.

Erosion: 1) Displacement of soil particles on the land surface due to such things as water or wind action. 2) The wearing away or eroding of material on the land surface or along channel banks by flowing water or wave action on shores.

Estidama: The established designation for sustainability of projects with Abu Dhabi and the region.

Estimate: A quantitative assessment of the likely amount or outcome. Usually applied to project costs, resources, effort, and durations and is usually preceded by a modifier (e.g. preliminary, conceptual, order-of-magnitude). An estimate may be expressed as a single number or as a range.

Exclusive Lane: 1) A lane or other facility that is fully grade separated or access controlled and is used only by a specified mode or vehicles at all times. 2) A lane reserved for buses/carpools only on a road, bridge, or tunnel that other traffic is restricted from using.

Expressway: An arterial roadway designed for relatively uninterrupted, high-volume mobility between areas, with full or partial control of access; may include a mixture of intersections (at grade) and interchanges (grade-separated).

Face of the Curb: The vertical or sloping surface on the roadway side of the curb.

Far-Side Stop: A transit stop that requires transit vehicles to cross the intersection before stopping to serve passengers.

Fill: Material used to raise the level of a low area.

Fill Slope: Slopes extending outward and downward from the shoulder or verge hinge point to intersect the natural ground line.

Flare: 1) The variable offset distance of a barrier to move it farther from the travelled way, generally in reference to the upstream end of the barrier. 2) Sloped surface that flanks a curb ramp and provides a graded transition between the ramp and the sidewalk. Flares bridge differences in elevation and are intended to prevent ambulatory pedestrians from tripping. Flares are not considered part of the accessible route.

Flow: 1) The movement of traffic. 2) A stream of water; movement of such things as water, silt, and/or sand; discharge; total quantity carried by a stream.

Flush Curb: A type of curb separating the travelled way and pedestrian realm where both are at the same level.

Flush Median: A paved median that is essentially level with the surface of the adjacent travelled way.

Freeway: The highest level of arterial. This facility is characterised by full control of access, high design speeds, and a high level of driver comfort and safety.

Front Slope: The side slope in a cut section created by connecting the shoulder to the ditch hinge point, downward and outward.

Frontage Road/Lane: An access road that generally parallels a major public roadway between the right-of-way of the major roadway and the front building setback line; provides access to private properties while separating them from the principal roadway.

Frontage Zone: The distance between the through zone and the building front or private property line in the pedestrian realm that is used to buffer pedestrians from window shoppers, appurtenances, and doorways.

Full Control of Access: Preference is given to through traffic by providing access connections by means of ramps with only selected public roads and by prohibiting crossings at grade and direct private driveway connections.

Functional Classification: A conventional system in which streets, roadways, and highways are grouped into classes according to the balance they strike between automobile mobility and land use access.

Furnishings Zone: The area of the pedestrian realm that provides a buffer between pedestrians and the edge zone, cycle track, parking lane, and/or vehicle travel lanes.

Gap: 1) The time, in seconds, for the front bumper of the second of two successive vehicles to reach the starting point of the front bumper of the first. 2) A break in the flow of vehicular traffic sufficiently long for a pedestrian to cross to the other side of the road or to a place of refuge.

Gap Acceptance: The process by which a minor-road vehicle accepts an available gap to manoeuvre.

Geographic Information System (GIS): A computer-based system that stores information based on geographical coordinates.

Geometric Design: Road design that deals with dimensions and relationships of such features as alignments, profiles, grades, sight distances, clearances, and slopes; distinguished from structural design, which is concerned with thickness, composition of materials, and load-carrying capacity.

Glare Screen: A device used to shield a driver's eye from the headlights of an oncoming vehicle.

Goods Vehicle: Goods vehicles have a range of uses but can generally be described as having a direct purpose of making or receiving goods for delivery. A goods vehicle is a facilitator of distribution. There are two distinct types of goods vehicle-a Heavy Goods Vehicle (HGV), which is over 3.5 tonnes, and a Light Goods Vehicle (LGV), which is below 3.5 tonnes

Gore: An area downstream from the shoulder intersection points at the divergence of two roadways.

Grade: 1) The rate of ascent or descent of a roadway, channel, or natural ground expressed as a percentage; the change in elevation per unit of horizontal length. 2) The finished surface of a canal bed, road bed, top of embankment, or bottom of excavation prepared for the support of such things as conduit, paving, ties, or rails.

Grade Crossing: The general area where a road and a railroad and/or light rail transit route cross at the same level, within which are included the tracks, road, and traffic control devices for traffic traversing that area.

Grade Separation: Any structure that provides a travelled way over or under another travelled way.

Gradient: Change of elevation, velocity, pressure, or other characteristics per unit length; slope.

Guardrail: 1) A type of longitudinal traffic barrier, usually flexible. 2) A traffic barrier used to shield obstacles from errant vehicles.

Guideline: Guidelines are not mandatory, but at considered the preferred practice in typical situations.

Guidestrip: Some type of raised material with grooves that pedestrians with vision impairments used for cane directional cues.

Gutter: That portion of the roadway section adjacent to the curb that is utilised to convey storm water runoff.

Headway: 1) The time, in seconds, between two successive vehicles as they pass a point on the roadway, measured from the same common feature on both vehicles, e.g. the front axles or the front bumper. 2) The time, usually expressed in minutes, between the passing of the front ends of successive transit units (vehicles or trains) moving along the same lane or track in the same direction. 3) The start to finish time of a fixed route.

High-Occupancy Toll Lane (HOT Lane): HOV facilities that allow lower-occupancy vehicles (e.g. solo drivers) to use these facilities in return for toll payments, which could vary by time of day or level of congestion.

High-Occupancy Vehicle (HOV): A motor vehicle carrying at least two occupants including the driver. An HOV could be a transit bus, a car pool, or any other vehicle that meets the minimum occupancy requirements.

High-Occupancy Vehicle Lane (HOV lane): An exclusive traffic lane or facility limited to carrying high-occupancy vehicles and certain other qualified vehicles.

High Speed: For geometric design purposes, high speed is defined as greater than 70 km/h.

Highway: A public way for purposes of vehicular travel, including the entire area within the right-of-way. In this *Manual*, the preferred term is road or roadway.

Highway Capacity: The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental and control conditions; usually expressed as vehicles per hour, passenger cars per hour, or persons per hour.

Impact Angle: For a longitudinal barrier, the angle between a tangent to the face of the barrier and a tangent to the vehicle's path at impact. For a crash cushion, the angle between the axis of symmetry of the crash cushion and a tangent to the vehicle's path at impact.

Interchange: A system of interconnecting roadways in conjunction with one or more grade separations, providing for the movement of traffic between two or more roadways on different levels.

Intersection/Junction: The general area where two or more roads join or cross, within which are included the roadway and roadside facilities for traffic movements in that area.

Intersection Angle: The angle between two intersection legs.

Intersection Leg: Anyone of the roadways radiating from and forming part of an intersection. The common intersection of two roadways crossing each other has four legs.

Interval: The part of a signal cycle during which signal indications do not change.

Island: A defined area between travel lanes for control of vehicular movements or for pedestrian refuge. Within an intersection area, a median, or an outer separation is considered an island.

Landing: 1) A level area at an intersection where vehicles stop prior to entering the intersection. 2) Level area of a sidewalk at the top or bottom of a ramp.

Lane: A strip of roadway used for a single line of vehicles.

Lane Line: A line separating two lanes of traffic travelling in the same direction.

Lay-By: A portion of the travelled way recessed into the pedestrian realm for taxis and loading so that travel lanes are not blocked.

Length of Need: Total length of a longitudinal barrier needed to shield an area of concern.

Level of Service: A qualitative measure describing operational conditions within a traffic stream based on service measures such as speed and travel time, freedom to manoeuvre, traffic interruptions, comfort, and convenience.

Local Road/Street: All public roads classified below the collector level primarily for access to residence, business, or other abutting property.

Longitudinal Barrier: A device whose primary function is to prevent penetration and to safely redirect an errant vehicle away from a roadside or median obstacle.

Loop: A one-way turning roadway that curves about 270 degrees to the right to accommodate a left turn from the through roadway. It may include provision for a left turn at a crossroad terminal.

Low Speed: For geometric design purposes, low speed is defined as 70 km/h or less.

Luminaire: A complete lighting unit consisting of a lamp or lamps together with the parts designed to distribute the light, to position and protect the lamps, and to connect the lamps to the electric power supply.

Major Road: The road at an intersection that normally carries the higher volume of vehicular traffic.

Median: The area between the two roadways or dual carriageways separating opposing directions of travel. It is measured from edge of travelled way to edge of travelled way and excludes turn lanes.

Median Barrier: A longitudinal barrier used to prevent an errant vehicle from crossing the road median.

Median Island: The centre of a street that physically separates the directional flow of traffic and can provide pedestrians with a place of refuge and reduce the crossing distance between safety points. It may also accommodate transit facilities. See also Refuge Island.

Median Opening: A gap in a median provided for crossing and turning traffic.

Midblock Crossing: A crossing point positioned within a block rather than at an intersection.

Midblock Stop: A transit stop located at a point midway between intersections.

Minimum Clearance Width: The narrowest point on a sidewalk or trail. A minimum clearance width is created when obstacles, such as utility poles or tree roots, protrude into the sidewalk and reduce the design width.

Minor Road: The road controlled by stop signs at a two-way stop-controlled intersection.

Multileg Intersection: An intersection with five or more legs.

Near-Side Stop: A transit stop located on the approach side of an intersection. The transit units stop to serve passengers before crossing the intersection.

Nonrecoverable Slope: A slope that is considered traversable but on which the errant vehicle will continue on to the bottom. Embankment slopes between IV:3H and IV:4H may be considered traversable, but nonrecoverable if they are smooth and free of fixed objects.

Offset: 1) Lateral distance from the edge of a travelled way to a roadside object or feature. 2) The difference, in seconds, between the start of individual green times and a specified time datum in a system of signalised intersections.

Off tracking: The difference in the paths of the front and rear tires of a vehicle as it negotiates a turn. The path of the rear tires of a turning vehicle does not coincide with that of the front tires.

On-Street Parking: Parking areas and parking spaces that are located on the street and/or in areas adjacent to the street within a right-of-way.

Operating Speed: The speed at which drivers are observed operating their vehicles during free-flow conditions.

Opposing Traffic: Vehicles that are traveling in the opposite direction. At an intersection, vehicles entering from an approach that is approximately straight ahead would be considered to be opposing traffic, but vehicles entering from approaches on the left or right would not be considered to be opposing traffic.

Outer Separation: The portion of an arterial road between the travelled ways of a roadway for through traffic and a frontage road.

Overpass: A grade separation where the subject road passes over an intersecting road or railroad.

Parallel Curb Ramp: A curb ramp design where the sidewalk slopes down on either side of a landing. Parallel curb ramps require users to turn before entering the road.

Parallel Parking: Parking that is parallel to the curb and path of travel.

Parking Bay: Parking that is provided in combination with curb extensions; also known as Parking Lay-By.

Parking Lane: An auxiliary lane primarily for the parking of vehicles.

Partial Control of Access: Preference is given to through traffic to a degree; access connections, which may be at grade or grade separated, are provided with selected public roads and private driveways.

Passenger Car: A motor vehicle, other than a motorcycle, designed for carrying 10 or fewer passengers and used for the transportation of persons.

Passenger Car Equivalent: The number of passenger cars displaced by a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions for use in capacity analysis.

Passing Sight Distance: 1) The length of road required for a vehicle to execute a normal passing manoeuvre as related to design conditions and design speed. 2) The visibility distance required for drivers to execute safe passing manoeuvres in the opposing traffic lane of two-way, two-lane roads.

Pathway: A general term denoting a public way for purposes of travel by authorised users outside the travelled way and physically separated from the roadway by an open space or barrier and either within the road right-of-way or within an independent alignment. Pathways include shared-use paths, but do not include sidewalks.

Pavement Markings: Markings set into the surface of, applied upon, or attached to the pavement for the purpose of regulating, warning, or guiding traffic. Also referred to as markings or traffic markings.

Peak Direction: The direction of higher demand during a peak commuting period. In a radial corridor, the peak direction has traditionally been toward the central business district in the morning and away from the central business district in the evening.

Peak Hour: That hour during which the maximum amount of travel occurs. It may be specified as the morning peak hour or the afternoon or evening peak hour.

Peak Hour Factor: 1) The hourly volume during the maximum-volume hour of the day divided by the peak I5-minute flow rate within the peak hour. 2) A measure of traffic fluctuation within the peak hour.

Pedestrian: A person afoot or in a wheelchair.

Pedestrian Access Route: A continuous, unobstructed path connecting all accessible elements of a pedestrian system.

Pedestrian Realm: The public area between the curb and the boundary of the right-of-way that is generally accessible to pedestrians.

Perpendicular Curb Ramp: A curb ramp design where the ramp path is perpendicular to the edge of the curb.

Physical Gore: A longitudinal point where a physical barrier or the lack of a paved surface inhibits road users from crossing from a ramp or channelized turn lane or channelized entering lane to the adjacent through lane(s) or vice versa.

Plans: The contract drawings that show the location, character, and dimensions of the prescribed work, including layouts, profiles, cross sections, and other details.

Point of Curvature (PC): The point where the tangent alignment ends and the horizontal curve begins.

Point of Intersection (PI): The point where two tangent lines meet.

Point of Tangency (PT): The point where the horizontal curve ends and the tangent alignment curve begins.

Post Speed Limit: A speed limit determined by law or regulation and displayed on Speed Limit signs.

Policy: A definite course of action or method of action selected to guide and determine present and future decisions.

Priority Lane: A lane providing preferential treatment to eligible vehicles.

Profile Grade: The trace of a vertical plane intersecting the top surface of the proposed wearing surface, usually along the longitudinal centreline of the roadbed. Profile grade means either elevation or gradient of such trace according to the context.

Project: The specific section of the road together with all appurtenances and construction to be performed thereon under the contract.

Project Specifications: The written documents that specify requirements for a project in accordance with service parameters and other specific criteria established by the contracting agency.

Protected Turn: The left or right turns at a signalised intersection that are made with no opposing or conflicting vehicular or pedestrian flow allowed.

Public Authority: A federal, municipal, or other local government instrumentality with authority to finance, build, operate, or maintain toll or toll-free facilities.

Quality: 1) The degree of excellence of a product or service. 2) The degree to which a product or service satisfies the needs of a specific customer. 3) The degree to which a product or service conforms to a given requirement.

Quality Assurance (QA): 1) The systematic utilization of performance requirements, design criteria, specifications, production control procedures, and acceptance plans for materials, processes, or products to ensure prescribed properties or characteristics. 2) All those planned and systematic actions necessary to provide confidence that a product, facility, or service will satisfy given requirements for quality.

Quality Control (QC): 1) The system of collection, analysis, and interpretation of measurements and other data concerning prescribed characteristics of a material, process, or product for determining the degree of conformance with specified requirements. 2) Those QA actions and considerations necessary to assess and adjust production and construction processes to control the level of quality being produced in the end product. 3) Encompasses all contractor/vendor operational techniques and activities that are performed or conducted to fulfil the contract requirements. 4) The sum total of activities that are performed or conducted to fulfil the contract requirements.

Radial Road: An arterial road leading to or from an urban centre.

Railroad: Consists of two steel rails that are held a fixed distance apart upon a roadbed. Vehicles, guided and supported by flanged steel wheels and connected into trains, are propelled as a means of transportation.

Railroad Grade Crossing: The general area where a road and a railroad cross at the same level, within which are included the railroad, roadway, and roadside facilities for traffic traversing that area.

Raised/Planted Median: A raised and/or planted median used to manage access and/or for aesthetic purposes.

Ramp: 1) A short roadway connecting two or more legs of an intersection or connecting a frontage road and main lane of a road. 2) A sloped transition between two elevation levels.

Recoverable Slope: A slope on which a motorist may, to a greater or lesser extent, retain or regain control of a vehicle. Slopes 1V:4H or flatter are generally considered recoverable.

Recovery Area: A clear zone that includes the total roadside border area, starting at the edge of the travelled way, available for safe use by errant vehicles.

Recreational Vehicle: A vehicle with light- or medium-duty chassis and suspension designed for recreational living or hauling.

Refuge Island: An island at or near a crosswalk or cycle path that aids and protects pedestrians and bicyclists who cross the roadway.

Rehabilitation: Work undertaken to restore the serviceability and to extend the service life of an existing bridge or road.

Rest Area: A roadside area with parking spaces separated from the roadway, provided for travellers to stop and rest for short periods. It may include drinking water, restrooms, tables and benches, information displays, and other facilities for travellers.

Resurfacing: The placing of one or more new courses on an existing surface.

Retaining Wall: A structure used to maintain an elevation differential between the roadway and top bank while at the same time preventing bank erosion and instability.

Right-of-Way: 1) Land, property or interest therein, usually in a strip, acquired for or devoted to transportation purposes. 2) The precedence in passing or proceeding accorded to one vehicle or person over another. 3) The legal power of passage over another person's land.

Road: A general term for denoting a public way for purposes of vehicular travel, including the entire area within the right-of-way, that is improved, designed, or ordinarily used for vehicular travel and parking lanes, but exclusive of the sidewalk, verge, or shoulder even though such sidewalk, verge, or shoulder is used by persons riding cycles or other human-powered vehicles. A road may be classified as single-carriageway or dual-carriageway (see carriageway).

Roadbed: The graded portion of a road within top and side slopes, prepared as a foundation for the pavement structure and shoulder.

Roadside: That area between the outside shoulder edge and the right-of-way limits. The area between roadways of a divided road may also be considered roadside.

Roadside Barrier: A longitudinal barrier used to shield roadside obstacles or nontraversable terrain features. It may occasionally be used to protect pedestrians or bystanders from vehicle traffic.

Roadway: 1) The portion of a road, including shoulders, for vehicular use. A divided road/dual carriageway has two or more roadways. 2) In construction specifications, the portion of a road within limits of construction.

Roadway Width: The roadway clear space between barriers, curbs or both.

Roundabout: A circular intersection with yield control of all entering traffic, channelized approaches with raised splitter islands, counter-clockwise circulation, and appropriate geometric curvature to encourage a travel speed on the circulating roadway of less than 20 km/h.

Rounding: The introduction of a vertical curve between two transverse slopes to minimise the abrupt slope change and to maximise vehicle stability and manoeuvrability.

Rumble Strip: A rough-textured surface constructed for the purpose of causing the tires of a motor vehicle driven over it to vibrate audibly as a warning to drivers.

Running Speed: The distance a vehicle travels divided by running time, in kilometres per hour.

Sediment: Fragmental material that originates from weathering of rocks and is transported by, suspended in, or deposited by water or air or is accumulated in beds by other natural agencies.

Semitrailer: 1) A freight trailer supported at its forward end by a truck tractor or another trailer and at its rearward end by attached axles. 2) A vehicle with or without motive power, designed for carrying persons or property and for being drawn by a motor vehicle and so constructed that some part of its weight and that of its load rests upon or is carried by another vehicle.

Service Flow Rate: The maximum hourly rate at which vehicles, cycles, or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a given time period (usually 15 minutes) under prevailing roadway, traffic, environmental, and control conditions while maintaining a designated level of service, expressed as vehicles per hour or vehicles per hour per lane.

Service Life: The period of time that the infrastructure element, vehicle, or equipment is expected to be in operation.

Shared Roadway: A roadway that is open to pedestrians, cyclists, and motor vehicle travel. This may be an existing roadway, road with wide curb lanes or road with paved shoulders.

Shared Use Path: A cycle path physically separated from motorised vehicle traffic by an open space or barrier and either within the road right-of-way or within an independent right-of-way. Shared use paths may also be used by pedestrians, skaters, wheelchair users, joggers, and other non-motorised users. Also referred to as cycle path or cycle track.

Shielding: The introduction of a barrier or crash cushion between the vehicle and an obstacle or area of concern to reduce the severity of impacts of errant vehicles.

Shoulder: The portion of the roadway contiguous with the travelled way primarily for accommodation of stopped vehicles for emergency use and for lateral support of base and surface course.

Sidewalk: A paved pathway paralleling a road intended for pedestrians. Also, see Pedestrian Realm.

Side Slope: A ratio used to express the steepness of a slope adjacent to the roadway. The ratio is expressed as vertical to horizontal (V:H)

Sight Distance: The length of road ahead that is visible to the driver.

Sign: A device conveying a specific message by means of words or symbols, erected for the purpose of regulating, warning, or guiding traffic.

Signal Head: An assembly containing one or more signal lenses that control a vehicular traffic or pedestrian movement.

Signal Phase: 1) The right-of-way, yellow change, and red clearance intervals in a cycle that are assigned to an independent traffic movement or combination of movements. 2) The part of the traffic signal cycle allocated to any combination of traffic movements receiving the right-of- way simultaneously during one or more intervals.

Single-Point Diamond Interchange (SPDI): A newer type of diamond interchange where the diagonal ramps are placed as close as possible paralleling the freeway, so that ramp traffic in effect meets at a single point on the surface road directly below (or above) the freeway.

Skew Angle: 1) The angle between the centreline of a support and a line normal to the roadway centreline. 2) The angle between the axis of support relative to a line normal to the longitudinal axis of the bridge. 3) The angle between the centrelines of two intersecting roadways where they intersect at less than 90 degrees.

Skid Resistance: The ability of the travelled surface to prevent the loss of tire traction, quantified by the frictional force between a locked tire and a pavement, which force resists motion.

Slip Ramp: 1) An angular connection between the through travel lanes and a parallel frontage road. 2) A type of at-grade access that can be used at the beginning or end of an HOV facility that provides an acceleration/deceleration taper.

Slope: The relative steepness of the terrain, expressed as a ratio or percentage. Slopes may be categorised as positive (backslopes) or negative (front slopes) and as parallel or cross slopes in relation to the direction of traffic.

Slope Ratio: An arithmetic expression of vertical and horizontal value relationships of a slope. Vertical values precede horizontal values.

Sloping Curb: Sloping curbs are designed to allow vehicles to readily cross the curb when the need arises. They generally have a height of 150 mm or less.

Spacing: The distance between two successive vehicles in a traffic lane measured from the same common feature of the vehicles.

Span: 1) The horizontal distance between vertical supports. 2) The horizontal width dimension of such things as a box, pipe-arch, or arch structure. 3) The horizontal distance between bridge piers or abutments.

Special Provisions: Additions and revisions to the standard and supplemental specifications applicable to an individual project.

Specifications: The compilation of provisions and requirements for the performance of prescribed work.

Speed: The rate of vehicular movement, generally expressed in kilometres per hour.

Speed-Change Lane: An auxiliary lane, including tapered areas, primarily for the acceleration or deceleration of vehicles entering or leaving the through travelled way.

Speed Limit: The maximum (or minimum) speed applicable to a section of road as established by law or regulation.

Speed Zone: A section of road with a speed limit that is established by law, but that might be different from a legislatively specified statutory speed limit.

Splitter Island: The raised island at each two-way leg between entering vehicles and exiting vehicles, designed primarily to deflect entering traffic.

Standard Drawings: Drawings approved for repetitive use, showing details to be used where appropriate.

Standard Specifications: A book of specifications approved for general application and repetitive use. The items in the standard specifications relate to or illustrate the method and manner of performing the work or describe the qualities and quantities of materials and labour to be furnished under the contract.

STEAM: Strategic Transportation Evaluation & Assessment Model of DMT

Steering Angle: The maximum angle of turn built into the steering mechanism of the front wheels of a vehicle along with the vehicle axle locations and articulation points. This maximum angle controls the minimum turning radius of the vehicle.

Stop Line: A solid, white pavement marking line extending across approach lanes to indicate the point at which a stop is intended or required to be made.

Street: 1) A public way for purposes of vehicular travel, usually including curb, to include the entire area within the right-of-way. 2) A paved public roadway in a built environment. It is a public parcel of land adjoining buildings in an urban setting.

Street (upper case "Street"): A low vehicle capacity 1+1 street (one lane in each direction). Anticipated low traffic volume and low speeds.

Street Realm: The area between curbs that encompass the motor vehicle travel lanes and may include parking lanes and transit lanes.

Streetscape: The elements of a street including the sidewalk, median, street furniture, trees, and open spaces that combine to form the street's character.

Structure: Any bridge, culvert, catch basin, drop inlet, retaining wall, cribbing, manhole, endwall, building, sewer, service pipe, underdrain, foundation drain and similar features, that may be encountered in the transportation system.

Superelevation: A tilting of the roadway surface to partially counterbalance the centripetal forces (lateral acceleration) on vehicles on horizontal curves. This is usually expressed as a rate.

Superelevation Rate: The rate of rise in cross section of the finished surface of a roadway on a curve, measured from the lowest or inside edge to the highest or outside edge travel lane or shoulder.

Surface Course: One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer is sometimes referred to as Wearing Course.

Surface Slope: The inclination of a surface, expressed as change of elevation per unit of slope length; the sine of the angle that the surface makes with the horizontal. The tangent of that angle is ordinarily used, with no appreciable error resulting except for the steeper slopes.

Swale: A wide, shallow ditch usually grassed or paved and without well-defined bed and banks. A slight depression in the ground surface where water collects and may be transported as a stream. Often vegetated and shaped so as not to provide a visual signature of a bank or shore.

Swept Path Width: The amount of roadway width that a vehicle covers in negotiating a turn that is equal to the amount of off tracking plus the width of the vehicle.

Tactile Warning: Change in surface condition providing a tactile cue to alert pedestrians with vision impairments of a potentially hazardous situation.

Tangent: Any straight portion of a roadway alignment.

Taper Area: An area characterised by a reduction or increase in pavement width to direct traffic.

Temporary Barrier: A barrier that is used to prevent vehicular access into construction or maintenance work zones and to redirect an impacting vehicle in order to minimise damage to the vehicle and injury to the occupants while providing worker protection.

Three-Leg Intersection: An intersection with three legs, where two roads join.

Through Road: Road or portion thereof on which vehicular traffic is given preferential right-of-way, and at the entrances to which vehicular traffic from intersecting roads is required by law to yield right-of-way to vehicles on such through road in obedience to either a stop sign or a yield sign, when such signs are erected.

Through Zone: The main area within the pedestrian realm where pedestrians travel.

T-Intersection: A three-leg intersection in the general form of a T.

Toe of Slope: The intersection of the fill slope, front slope, or back slope with the natural ground line or ditch bottom.

Tractor/Trailer Angle: The angle between adjoining units of a tractor/semitrailer when the combination unit is placed into a turn. This angle is measured between the longitudinal axes of the tractor and trailer as the vehicle turns.

Traffic: The movement of vehicles, pedestrians, ships, or planes through an area or along a defined route.

Traffic Assignment: A process by which trips described by mode, purpose, origin, destination, and time of day are allocated among the paths or routes in a network according to one of a number of flow-distribution models.

Traffic Barrier: A device used to prevent a vehicle from striking a more severe obstacle or feature located on the roadside or in the median or to prevent crossover median crashes. There are four classes of traffic barriers: roadside barriers, median barriers, bridge railings, and crash cushions.

Traffic Control Device: A sign, signal, marking, or other device used to regulate, warn, or guide traffic, placed on, over or adjacent to a road open to public travel, pedestrian facility, or shared-use path by authority of a public agency or official having jurisdiction.

Traffic Island: A defined area between traffic lanes for control of vehicle movements or for pedestrian refuge. Within an intersection, a median or an outer separation is considered an island.

Traffic Sign: A device mounted on a fixed or portable support whereby a specific message is conveyed by means of words or symbols, officially erected for the purpose of regulating, warning, or guiding traffic.

Traffic Signal: Any power-operated traffic control device, other than a barricade warning light or steady burning electric lamp, by which traffic is warned or directed to take some specific action.

Traffic Volume: The number of persons or vehicles passing a point on a lane, roadway, or other travelled way during some time interval, often one hour, expressed in vehicles, bicycles, or persons per hour.

Trailer: A vehicle designed for carrying persons or property and drawn by a motor vehicle that carries no part of the weight and load of the trailer.

Transition: A section of barrier between two different barriers or, more commonly, where a roadside barrier is connected to a bridge railing or to a rigid object such as a bridge pier.

Transit Way: The term used to describe an HOV lane or facility. In some cases, it refers to bus-only facilities, but in other cases, it may be used on a facility open to all HOVs.

Travelled Way: The portion of the roadway for the movement of vehicles, exclusive of shoulders.

Travel Speed: The speed over a specified section of a road, being the distance divided by travel time.

Travel Time: The time of travel, including stops and delays, except those off the travelled way.

Traversable Slope: A slope from which a motorist will be unlikely to steer back to the roadway, but may be able to slow and stop safely. Slopes between IV:3H and IV:4H generally fall into this category.

Truck: 1) The assembly of parts consisting of wheels and axles with necessary springs and structural members that support the main body of a rail car at each end. 2) A wheeled road freight vehicle also referred to as goods vehicle.

Truck Apron: The optional outer, mountable portion of the central island of a roundabout between the raised, nontraversable area of the central island and the circulating roadway.

Truck Tractor: A motor vehicle designed for drawing other vehicles, but not for a load other than a part of the weight of the vehicle and load drawn.

Trumpet Interchange: A three-way interchange with no crossing movements, featuring one 270-degree loop ramp opposite the terminating roadway and a semi directional ramp following the loop to the outside.

Truncated Domes: Small domes with flattened tops used as tactile warning at transit platforms and at other locations where a tactile warning is needed.

Turning Movement: The traffic making a designated turn at an intersection.

Turning Path: The path of a designated point on a vehicle making a specified turn.

Turning Radius: The path that a vehicle takes during a turn.

Turning Track Width: The radial distance between the turning paths of the outside of the outer front tire and the outside of the rear tire that is nearest the centre of the turn.

Turn Lane: An auxiliary lane adjoining the through travelled way for speed change, storage, and turning.

Turnout: A short segment of a lane, usually a widened, unobstructed shoulder area, added to a two-lane, two-way road, allowing slow-moving vehicles to leave the main roadway and stop so that faster vehicles can pass.

Underpass: A grade separation where the subject road passes under an intersecting road or railroad. Also referred to as Undercrossing.

Undivided Roadway: A roadway that does not have a physical barrier (e.g. depressed median, CMB median) between opposing traffic lanes.

Urban Area: Existing and planned areas as defined in the maps contained in the Plan Abu Dhabi 2030, Plan Al Ain 2030, and Plan Al Gharbia 2030.

Value Engineering: An analysis of materials, processes, and products in which functions are related to cost and from which a selection may be made for the purpose of achieving the required function at the lowest overall cost consistent with the requirements for performance, reliability, and maintainability. Also referred to as Value Analysis.

Vehicle: 1) An assembly of one or more units coupled together for travel on a roadway; vehicles include one powered unit and may include one or more unpowered full trailer or semitrailer units. 2) Every device in, upon, or by which any person or property can be transported or drawn upon a road, except trains and light rail transit operating in exclusive or semi exclusive alignments. Light rail transit operating in a mixed-use alignment, to which other traffic is not required to yield the right-of-way by law, is a vehicle.

Verges: The part of the cross section that acts as a buffer zone between the edge of the pavement (curb or back of shoulder) and the surrounding physical features.

Vertical Clearance: Minimum unobstructed vertical passage space required along a sidewalk or trail. Vertical clearance is often limited by obstacles such as building overhangs, tree branches, signs, and awnings.

Vertical Curb: A longitudinal element, typically concrete, placed at the roadway edge for delineation, to control drainage, to control access, etc. Vertical curbs may range in height between 150 mm t0 300 mm with a face no steeper than 6:1 V:H

Vertical Point of Curvature (VPC): The point at which a tangent grade ends and the vertical curve begins.

Vertical Point of Intersection (VPI): The point where the extension of two tangent grades intersect.

Vertical Point of Tangency (VPT): The point at which the vertical curve ends and the tangent grade begins.

Very Low-Volume Local Road: A road that is functionally classified as a local road and has a design average daily traffic volume of 400 vehicles per day or less.

Walkway: A facility provided for pedestrian movement and segregated from vehicular traffic by a curb, or provided for on a separate right-of-way.

Warrants: 1) A document that provides guidance to the designer in evaluating the potential safety and operational benefits of traffic control devices or features. Warrants are not absolute requirements; rather, they are a means of conveying concern over a potential traffic hazard. 2) The criteria by which the need for a safety treatment or improvement can be determined.

Weaving: The crossing of two or more traffic streams travelling in the same direction along a significant length of road without the aid of traffic control devices (except for guide signs).

Weaving Segment: A length of roadway over which traffic streams cross paths through lane-changing manoeuvres, without the aid of traffic signals, formed between merge and diverge points.

Weigh-in-Motion (WIM): A set of sensors and supporting instruments that measures the presence of a moving vehicle and the related dynamic tire forces at specified locations with respect to time; estimates tire loads, speed, axle spacing, vehicle class according to axle arrangement, and other parameters concerning the vehicle; and processes, displays and stores this information.

Weigh Station: A roadway facility for scales to weigh a commercial vehicle and its contents; it may be mobile or fixed and may include a building, ramps and parking.

Wheel Track: A line or path followed by the tire of a road vehicle on a travelled surface.

Work Zone: An area of a roadway with construction, maintenance, or utility work activities, typically marked by signs, channelizing devices, barriers, pavement markings, work vehicles, or combination thereof. It extends from the first warning sign or high-intensity rotating, flashing, oscillating, or strobe lights on a vehicle to the END ROAD WORK sign or the last temporary traffic control device.

Y Intersection: A three-leg intersection in the general form of a Y.

Geometric Qualifying Words

Shall, require, will, must: A mandatory condition. Designers are obligated to adhere to the criteria and applications presented in this context or to perform the evaluation indicated.

Should, recommend: An advisory condition. Designers are strongly encouraged to follow the criteria and guidance presented in this context, unless there is reasonable justification not to do so.

May, could, can, suggest, consider: A permissive condition. Designers are allowed to apply individual judgment and discretion to the criteria when presented in this context. The decision will be based on a case-by-case assessment.

Desirable, preferred: An indication that the designer should make every reasonable effort to meet the criteria and that the designer should only use a "lesser" design after due consideration of the "better" design.

Ideal: Indicating a standard of perfection (e.g. traffic capacity under "ideal" conditions).

Minimum, maximum, upper, lower (limits): Representative of generally accepted limits within the design community, but not necessarily suggesting that these limits are inviolable.

Practical, feasible, cost-effective, reasonable: Advising the designer that the decision to apply the design criteria should be based on a subjective analysis of the anticipated benefits and costs associated with the impacts of the decision. No formal analysis (e.g. cost-effectiveness analysis) is intended, unless otherwise stated.

Possible: Indicating that which can be accomplished. Because of its rather restrictive implication, this word is rarely used in this *Manual* for the application of design criteria.

Significant, major: Indicating that the consequences from a given action are obvious to most observers and, in many cases, can be readily measured.

Insignificant, minor: Indicating that the consequences from a given action are relatively small and not an important factor in the decision-making for design.

Warranted, justified: Indicating that some well-accepted threshold or set of conditions has been met. Note that, once the warranting threshold has been met, this is an indication that the design treatment should be considered and evaluated – not that the design treatment is automatically required.

Standard: Indicating a design value that cannot be violated without severe consequences.

Acronyms

AADT: annual average daily traffic
AASHTO: American Association of State Highway and Transportation Officials
AACM: Al Ain City Municipality
ADM: Abu Dhabi City Municipality
ADT: average daily traffic
CSS: context sensitive solutions
DHV: design hourly volume
DMT: Department of Municipalities and Transport
DRM: Dhafra Region Municipality**DSD:** decision sight distant
FHWA: US Federal Highway Administration
HCM: Highway Capacity Manual
HOV: high-occupancy vehicle
ISD: intersection sight distance
ITE: Institute of Transportation Engineers
ITS: intelligent transportation systems
km: kilometre(s)
km/h: kilometres per hour
KSA: Kingdom of Saudi Arabia
LOS: level of service
m: metre(s)
mm: millimetre(s)
MUTCD: Manual on Uniform Traffic Control Devices
NCHRP: US National Cooperative Highway Research Program
NMU: non-motorised users
PC: point of curvature
PI: point of intersection of tangents
PSD: passing sight distance
PT: point of tangency
PTW: powered two wheelers
R: radius
ROW: right of way
RSA: road safety audit
SSD: stopping sight distance
TRB: US Transportation Research Board
US: United States
UAE: United Arab Emirates
DMTUSDOT: United States Department of Transportation
VPC: vertical point of curvature
VPH or vph: vehicles per hour
VPI: vertical point of intersection
VPT: vertical point of tangency
VRS: vehicle restraint systems
:
ZOI: zone of intrusion

1 INTRODUCTION TO ROAD DESIGN

1.1 Overview

In 2010, the Abu Dhabi Department of Transport commenced with the “Unifying and Standardizing of Road Engineering Practices” Project. The objective of the project was to enhance the management, planning, design, construction, maintenance, and operation of all roads and related infrastructures in the Abu Dhabi Emirate and ensure a safe and uniform operational and structural capacity throughout the road network.

To achieve this objective a set of standards, specifications, guidelines, and manuals were developed in consultation with all relevant authorities in the Abu Dhabi Emirate including internal stakeholders in the Department of Municipalities and Transport (DMT)DMT. In future, all authorities or agencies involved in roads and road infrastructures in the Emirate shall exercise their functions and responsibilities in accordance with these documents. The purpose, scope, and applicability of each document are clearly indicated in each document.

It is recognised that there are already published documents with similar objectives and contents prepared by other authorities. Such related publications are mentioned in each new document and are being superseded by the publication of the new document, except in cases where previously published documents are recognised and referenced in the new document. In this case, the new document complements the previously published documents. For example, this *Manual* along with the DMT *Abu Dhabi Urban Street Design Manual* contains guidance for the planning and concept design of urban streets.

1.2 Objectives

The guidance supplied in this document, *Road Geometric Design Manual*, is based on established international practices (in particularly, American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (1) and is supplemented by recent research in roadway design; AustRoads *Guide to Road Design* (2), United Kingdom *Design Manual for Roads and Bridges* (3), Abu Dhabi Urban Planning Council *Abu Dhabi Urban Street Design Manual* (4), and other roadway documents.

The *Road Geometric Design Manual* has been prepared to provide uniform practices for Abu Dhabi design studies, design reports, and roadway plans prepared by consultants. The designer should meet all criteria and practices presented in the *Manual*. Designers must ensure roadway designs meet operational and Road Safetyrequirements) while preserving the aesthetic, historic, or cultural resources of Abu Dhabi. The guidance provided in this document should be supplemented with good engineering knowledge, experience, and sound judgment.

The *Road Geometric Design Manual* will be updated regularly as new data and experience with best practices become available. Abu Dhabi *Road Geometric Design Manual* is effective from the date of issue for all Abu Dhabi roadway projects.

1.3 Application

The guidance presented in this document is intended for the design of new construction and major reconstruction of rural roads and urban streets in Abu Dhabi and to ensure uniformity of roadway designs within the Emirate of Abu Dhabi. Design guidance is provided for rural and urban freeways, rural arterials, rural collectors, and rural local roads. Design guidance is also provided for boulevards, avenues, streets, and access lanes for streets within urban areas. For the purposes of this Manual, urban streets are all streets within the existing and planned urban areas of the Emirate. Urban areas are defined in the maps contained in Plan Abu Dhabi 2030, Plan Al Ain 2030, and Plan Al Gharbia 2030.

The criteria in this Manual, generally, does not apply to rehabilitation, restoration, and resurfacing (3R) projects, spot improvements projects, or maintenance projects.

In situations of exceptional difficulty, it may be possible to overcome them by adopting Departures from Minimum Desirable Standards. Proposals to adopt different critiera than stated in this Manual must be submitted to the Department of Transport for approval before incorporation into a design layout to ensure that road safety is not significantly compromised.

1.3.1 Departures from Standards

A Departure from Standards relates to a design feature which does not meet the design standards presented in this Roadway Design Manual. Occasionally these departures are justified, but it is important that each Departure from Standard is documented and approved in writing, prior to planning acceptance.

The request for approval of these departures shall be in the form of a Departure from Standards Request. This request sheet shall be presented to DMT and other relevant stakeholders for written approval. The request sheet shall include the following information:

- Proposed project.
- Existing roadway description.
- Proposed Departure from Standards.
- Additional cost required to comply with Standards.
- Incremental improvements.
- Supportive data.

A detailed description of the items required in the Departure from Standards Request is included on the following pages.

1.3.2 Departure from Standards Request

1. Proposed Project

A. ***Project Description –*** Brief description of the project. This will include the type of project and/or major elements of work to be done, such as safety or operational improvement, roadway widening, rehabilitation, reconstruction, etc. The geographical project limits and length will be provided.

B. ***Proposed Project Cost -*** Include an estimate of the proposed project cost segregated into the major elements, including roadway, structures, right-of-way, utility relocation, environmental mitigation, etc.

2. Existing Roadway Description

Describe the existing roadway features relevant to the proposed Departure from Standards. This may include the widths of lanes, shoulders, median, clear zones and structures, horizontal and vertical alignment and clearances, design speed, sight distance, grades, cross slope, superelevation, etc.

If relevant, provide a similar brief description of adjacent existing roadway segments, noting any existing non-standard features.

3. Proposed Departure from Standards

A. State the specific design standard(s) which are not being met and refer to the relevant RDM section/clause number.

B. Describe the proposed Departure from Standards or the existing departure which is proposed to be maintained. For a proposed departure, state whether it is an improvement over the existing condition. Describe proposed improvements that would qualify as safety enhancements over the existing condition, such as median barrier, guardrail upgrade, flattening slopes, correcting superelevation, eliminating roadside obstructions, drainage, street lighting, signage, etc.

C. Provide a thorough brief justification for the departure. Reasons for granting departures include a combination of excessive cost, right-of-way impacts and/or environmental impacts. Supportive factors have included low accident frequency, local opposition and consistency with adjacent roadway segments.

4. Additional Cost Required to Comply with Standards

Provide a realistic estimate of the additional cost required to meet the design standard for which the proposed departure is requested.

5. Incremental Improvements

Discuss other practical alternatives that are intermediate in scope and cost between the proposed project (requiring this Departure from Standards) and the full, standard solution. Provide enough information on costs versus benefits, right-of-way and environmental impacts, etc., to explain why none of the incremental alternatives are recommended. These alternatives should normally be investigated prior to requesting a departure.

6. Supportive Data

A. ***Traffic Data -*** Provide both ADT and DHV (design hourly volumes) traffic information, quoted for the proposed future design year.

B. ***Accident Analysis -*** Safety is of primary importance when considering design approval for Departures from Standards. If relevant, include an accident data analysis to identify prevalent accident types and causes, plus an evaluation of the effect of the requested departure on accident types and frequencies.

C. ***Attachments -***

1. Provide a location plan for the project.

2. Provide plan sheets, cross sections, profiles and/or special details to clearly illustrate the proposed departure.

3. Attach pertinent letters, resolutions, meeting minutes, studies, etc., which further develop or clarify the proposed departure.

1.4 DESIGN CONCEPT DEVELOPMENT

1.4.1 Transportation Planning

1.4.1.1 Introduction

The pre-design process involves the collection of existing data from DMT, other government departments, utility agencies/authorities, landowners, and field surveys. This data becomes the foundation for project road and bridge design. The Consultant is responsible for all data collection.

1.4.1.2 Internal Roads And Infrastructure Directorate

IRID is the lead department from which all road and bridge projects are initiated and approved.

The Consultant will work with assigned staff to develop the project scope as per the Consultant Procedure Manual and identify applicable design criteria from the Roadway Design Manual. The Consultant is expected to develop the project by proper application of DMT policies, procedures and standards.

1.4.1.3 Town Planning Sector

The Town Planning Sector (TPS) is divided into the following:

1) Urban Planning Division (UPD).
2) Spatial Data Division (SDD).
3) Construction Permits Division (CPD).

The UPD (Planning Section) is responsible for the development and maintenance of the Master Plan and planning layouts. The Master Plan is the base document from which the project's roadway classifications are assigned. Roadway design standards are identified for each roadway classification

The planning layouts are used to identify the existing and proposed land use and development intensity.

The UPD (Utilities Section) is responsible for the development and approval of all service reservations.

1.4.1.4 Mapping

For standards, specifications and procedures, refer to the latest TPS (SDD) requirements.

1.4.1.5 General

Current, accurate base mapping is an essential tool in transportation planning and design. The specific mapping requirements depend on the length and complexity of the project and its location, either urban or rural. Aerial mapping is normally the most useful and cost-effective medium for larger projects. Ground topographical surveys are used for smaller projects, especially in urban areas and to supplement aerial mapping at specific locations where more detail and accuracy is needed.

Three types of aerial maps are used in the planning and design phases of roadway and bridge projects:

Uncontrolled Aerial Photography – These maps are produced directly from the aerial photographs that normally cover large areas at a reduced scale. The maps are generally used in route location studies to define transportation corridors and alternative alignments. The contact prints from the aerial photography are assembled to form a photo mosaic of the area under study to reduce distortion.

Controlled Aerial Photography - Prior to the flight, horizontal and vertical ground control points are set and marked in the field. These points are used to control photo mosaic products that are significantly more accurate and can be prepared at a specific scale. These maps can be used at larger scales for preliminary engineering activities including Design Concept Reports.

Topographic (Aerial or Mobile) Mapping - These consist of topographic maps compiled from airborne LiDAR data to capture large amounts of data over large areas and ground based LiDAR (fitted to a vehicle) to provide a greater amount of detail in specific areas. Data from LiDAR can be imported into CAD packages, combined with visual imagery or viewed directly in software packages like RiSCAN. This mapping can be used for both design concept development and final design and should be limited to the broad roadway corridor.

The Consultant is responsible for providing base mapping for design concept development. Specific requirements will be identified in the Consultant’s scope of work. Existing aerial and topographic maps may be available and suitable for use in consultation with DMT. DMT and TPS maintain a limited library of existing mapping which the Consultant may review for background information.

Mapping scales and contour intervals generally suitable for the intended purpose are shown in Table 1.1.

1.4.1.6 Topographic Mapping

Topographic maps for a specific project shall be prepared in accordance with the following:

Survey Control / Field Surveys - The requirements for surveys are included in Survey Control/Field Survey.

Drafting Standards - Mapping features and symbology will be prepared in accordance with the latest CAD Standards, supplemented by the standard symbols shown in Figures 1-1, 1-2 and 1-3.

Primary Control Points - All primary control points for mapping which were established during the initial field survey will be shown on the maps in their proper locations and with the appropriate symbol, identification number and elevation. A tabulation of the primary control points shall also be shown in the original survey notebook. The tabulation will show the identification number, coordinates and elevation of the point.

Supplemental Control Points - All supplemental control points established for controlling aerial photography will be shown on the maps. These include wing points, analytically bridged points, and aerial photo centres. See Figure 1-1.

Planimetric Features - Natural and manmade features, spot elevations, topographic features and relevant political subdivision lines shall be plotted on the maps as shown in Figures 1-2 and 1-3.

Coordinate Grid - Coordinate grid ticks shall be shown on the maps at intervals to suit the drawing.

North Point - A north point shall be placed on each map sheet. The north point shall be oriented so that north points to the top or to the right of the map sheet. Cut lines shall also be labelled so that each sheet may be joined accurately to adjacent sheets.

Map Index - A sheet index diagram shall be prepared for each mapping project. This diagram shall show the position and relationship of each sheet to adjacent sheets. A title block is also required and shall be placed on each sheet.

Table 1-1: Map scales and contour intervals for highway development

Purpose	Scales	Interval (m)
Route Location Studies: Mountainous Rolling to Flat	 1 : 5,000 (max) 1 : 5,000	 5 2
Preliminary Design (DCR): Rural Urban	 1 : 1,250 1 : 1,250	 2 2
Rural Design	1 : 1,250	0.5
Urban Design	1 : 500	0.5
Detailed Site Design	1 : 100 1 : 250	0.5 0.5

1.4.1.7 Project Limits

DMT will determine the limits of the project. Typically, the limits include the roadway/bridge, medians, sidewalks, parkways, and roadside improvements that enhance the appearance, maintainability and safety characteristics of the project. The project limits may also be determined by phased implementation considerations.

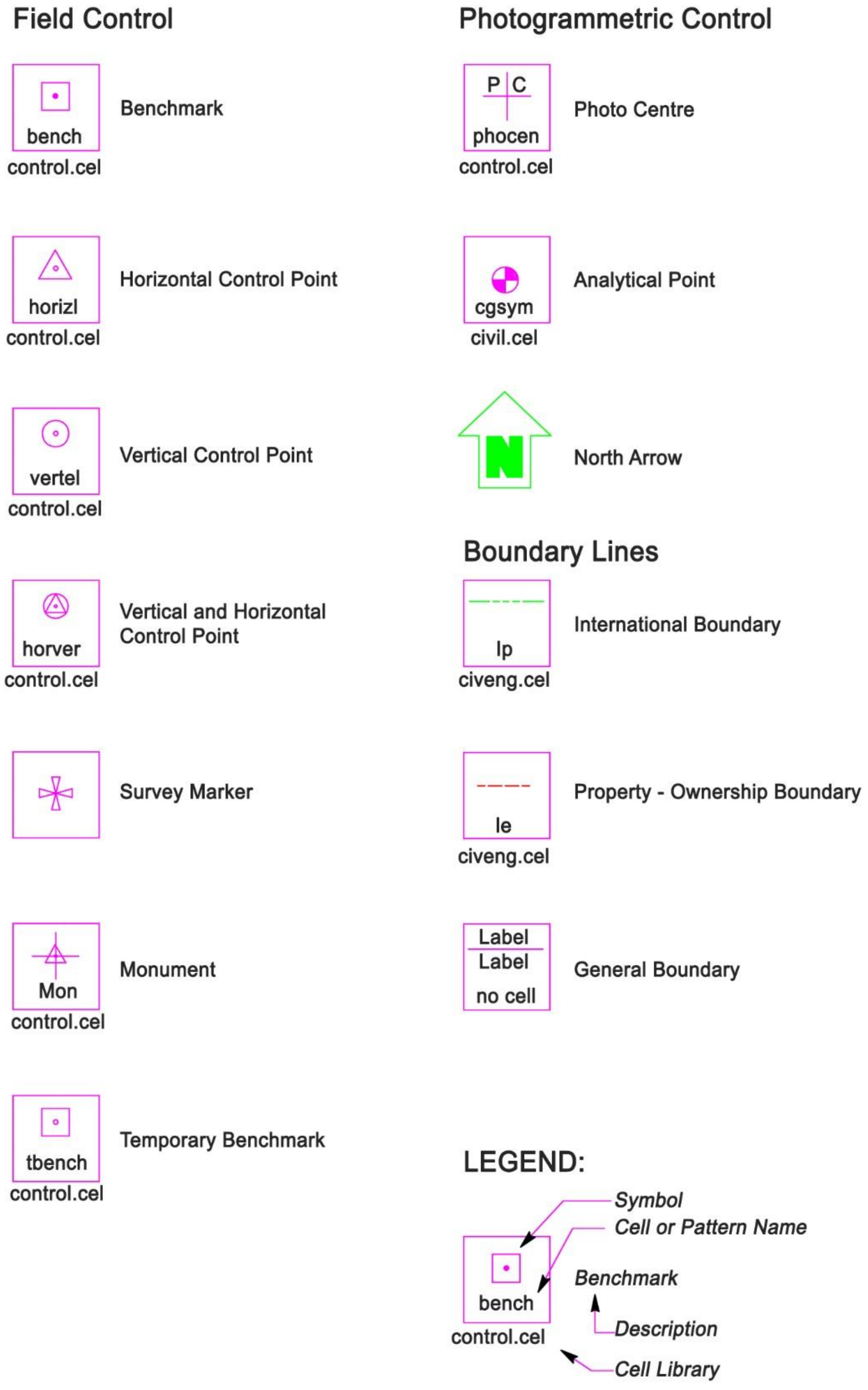


Figure 1-1: Standard Mapping Symbols - Boundaries and Monuments

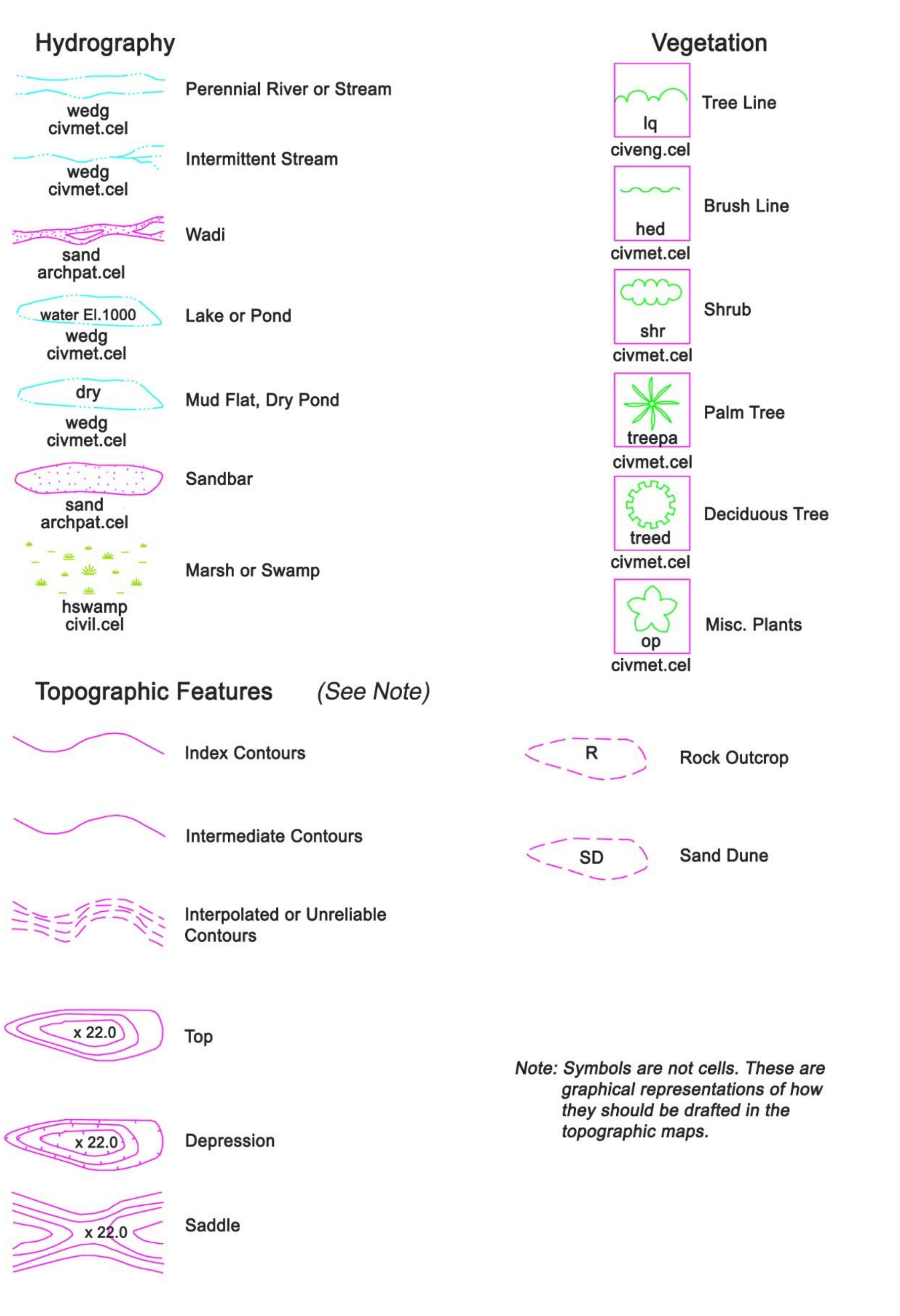


Figure 1-2: Standard Mapping Symbols - Natural Planimetric Features

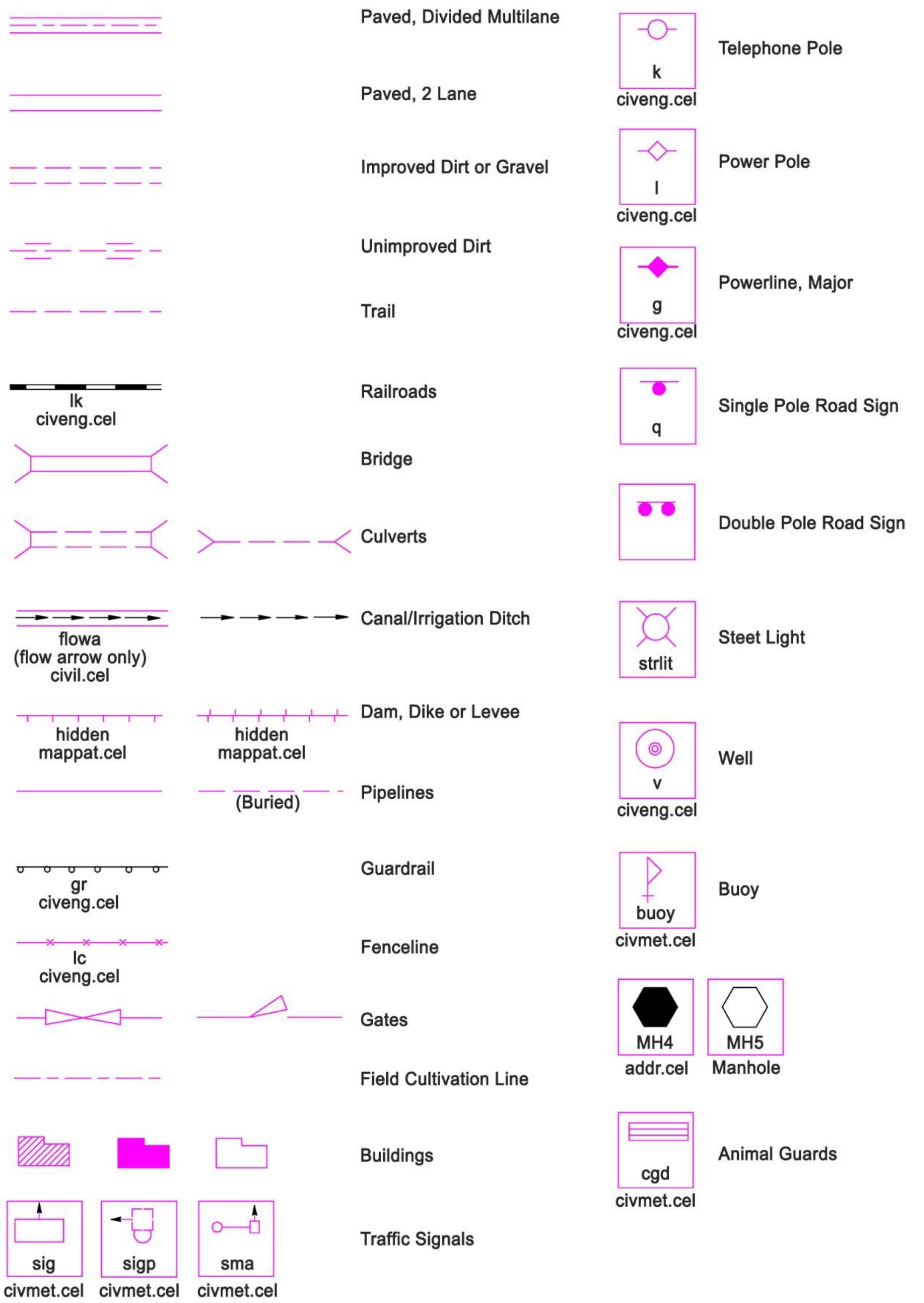


Figure 1-3: Standard Mapping Symbols - Manmade Planimetric Symbols

1.4.1.8 Project Identification And Numbering

DMT will assign the Title and Number for each individual roadway and bridge project. The Consultant will include this information on all drawings, reports, correspondence, calculations and other design documentation associated with the subject project.

1.4.1.9 Inter-Departmental Coordination

Throughout the development of the project, coordination with DMT as well as other government departments is essential. The Consultant is expected to identify the requirements of the involved government departments, and ensure that the project design addresses these requirements. Table 1.2 below lists the agency or authority responsible for transportation related functions.

Table 1-2: Municipal Agencies

Function	Agency/Authority
Road/Bridge Construction	DMT – Internal Roads and Infrastructure Directorate (IRID)
Planning	DMT DMT - Town Planning Sector (TPS) Department of Municipalitiesand Transport (DMT)
Utilities	DMT - Town Planning Sector (TPS) DMT
Parking	ADM (IRID) Department of Municipaitiesl and Transport (DMT)
Right-of-Way	DMT - Town Planning Sector (TPS)
Landscape and Public Realm	DMT – Parks & Recreation Facilities Division (PRFD)
Public Transportation	Department of Municipal Affairs and Transport (DMT)
Road Safety Audit	Municipalities Road Safety Teams

1.4.2 Environmental Factors Influencing Design

1.4.2.1 Introduction

There are a number of important environmental factors that influence the design of all roadway and bridge projects. These factors are both natural and man-made and have been divided into two major categories; Socio Economic/Community Resource Data and Natural/ Environmental Resource Data. The identification of these resources enables the project to be developed to avoid and/or minimize impact on these resources to the greatest extent practicable.

This will contribute significantly to public acceptance and the ultimate success of a project. The following sections describe the various environmental factors that comprise each of the two categories. It is the Consultant’s responsibility to assess each factor and develop a functional and compatible design.

The Consultant is responsible for ensuring that an accredited Environmental Consultant is used to study the environmental aspects of the project, facilitating the issuance of the environmental permit.

1.4.2.2 Socio Economic / Community Resource Data

The Consultant shall consider each of the following factors as part of the development of project design. The goal is to develop a functional design that accommodates, maintains or enhances the integrity of each socio economic and community resource with minimal disruption. To assist with the planning involved with the development of the design, the Consultant should map all resources that are capable of being placed onto a map.

Consultants are to ensure that the pedestrian environment is ‘accessible to all’, not only critical to meeting the access needs of individual disabled people, but contributing towards social inclusion and quality of life to a much wider section of the population. In this context ‘accessible to all’ means continuous and level sidewalks and paths, disabled parking/access, ramps to sidewalks and buildings, all of which must be clear of unnecessary obstructions.

The following factors must also ensure that the needs of pedestrians, cyclists, children and disabled people are carefully considered.

1.4.2.3 Land Use

The project plans must accommodate existing and future land use to the full extent possible. The Consultant is required to provide adequate parking and access to adjacent land uses, commensurate with the type of land use and the roadway classification. The roadway volumes used to determine the “level of service” (existing and 20-year projection) must include the trip generation associated with the adjacent land uses.

In urban areas, Plan Abu Dhabi 2030 (Urban Structure Framework Plan), published by the Urban Planning Council (DMT) is the primary document used to identify the types and locations of designated land uses. In rural areas, where the land usage is less defined, the Consultant must conduct a field survey of the existing land uses adjacent to the project. The aforementioned information, combined with the field survey data, will then be used to identify potential improvements to be designed as part of the roadway project.

In rural areas, formal information regarding land use may not be available. In these cases, the current land use is typically agricultural and will remain as agricultural unless there is information stating otherwise.

1.4.2.4 Growth Projections

The Consultant is to liaise with all concerned authorities to establish growth projections applicable to the area of the project. This will not only include DMT and TPS master planning projections, but will also take into consideration information obtained from developers working in the vicinity of the proposed scheme.

1.4.2.5 Public Services

The development of all road and bridge projects typically affects many public services. This can result from encroachment of the improvement project beyond the existing roadway, sidewalk, and bridge. As such, pre-design coordination with public services is required to incorporate design approaches and construction phasing that minimizes the project impact.

The Consultant is responsible for identifying all public services which may be affected by the project. In addition, the Consultant is also responsible for compiling all relevant design requirements from the affected public services and incorporating these parameters into the project design. It is the Consultant's responsibility to assure DMT that the design and construction phasing meets the approval of the affected public services.

Table 1-3: identifies the various public services and the responsible agency/authority for each.

Service	Agency/Authority
Road/Bridge Construction	DMT – IRID DMT
Landscape and Public Realm	DMT - PRFD
Police	General Headquarters of Abu Dhabi Police Directorate of Traffic and Patrols
Fire	Abu Dhabi Civil Defence
Security	Abu Dhabi Civil Defence Abu Dhabi Police
Schools	ADM – TPS DMT of Education Abu Dhabi Education Council (ADEC)
Sanitation	Health Authority of Abu Dhabi (HAAD) Abu Dhabi Sewerage Services Company (ADSSC)
Waste	Center of Waste Management
Parking	ADM – IRID DMT
Recreational	DMT – TPS DMT - PRFD
Navigable Waters	***Critical Infrastructure and Coastal Protection Authority***
Mail Service	Emirates Post Group
Public Transportation	DMT
National Railway and Metro Rail System	Etihad Rail – National Railway DMT – Abu Dhabi Metro Rail System
Mosques	General Authority of Islamic Affairs and Endowments Mosque Development Committee
Hospitals	Health Authority of Abu Dhabi (HAAD)
Archaeology	Abu Dhabi Authority for Culture & Heritage

1.4.2.6 Schools

Schools are an important national resource. The design shall accommodate and preserve sufficient access to all facilities that are affected by project design. Therefore, the Consultant is expected to adapt the project's design to accommodate each school's needs.

The Consultant should liaise with the DMT School Safety Zone Coordinator, DMT Traffic Services Section and authorized staff and/or management of any affected schools to establish the requirements.

Initial concept designs should study options to achieve better circulation, accessibility and way-finding. Operational requirements of the facility should be well analyzed and alternatives should be produced accordingly, taking into consideration the integration of different modes of transport and pedestrian safety (especially due to the higher risks associated with children) and as further detailed below:

1. Operational characteristics of the school should be thoroughly discussed with the concerned people, and observed at the site, including beginning and end of a school day for different grades, drop-off/pick-up locations for parents, drop-off/pick-up waiting time, staff parking requirements, parking and drop-off/pick up locations for school buses, access routes to main roads, etc.
2. Cycle and pedestrian routes for non-motorized students/parents.
3. Safe pedestrian crossing locations.

Parking supply should be in accordance with the guidelines on this type of facility, but any anticipated deficiency should be highlighted at the early stages of the design process.

For each school, there are a number of factors that must be considered in the project design. These include:

- school bus traffic
- crosswalks
- school yard fencing
- parking
- drop-off/pick-up waiting time
- landscaping
- noise attenuation (i.e., insulated windows, sound-proof walls)
- other safety improvement relocation of affected structures, as necessary
- affects on potential school expansion

In the case of new school site development, the DMT Master Plan and Abu Dhabi Education Council shall be consulted to identify these sites within and/or adjacent to the project limits.

As with all other adjacent property improvements, the Consultant is required to provide plans which can be used to construct the necessary improvements either in conjunction with the roadway/bridge project or as a separate project. This is intended so that construction can be undertaken on the school sites during scheduled school closures or outside of normal school working hours.

1.4.2.7 Mosques

Mosques are extremely important to the Islamic faith and cannot be relocated or impacted in any way. The Consultant shall identify all existing and proposed Mosques within close proximity to a proposed project. The project design shall avoid impact to Mosques and shall accommodate and preserve sufficient access to these sites. Construction works cannot be undertaken during prayer times under any circumstances.

1.4.2.8 Malls

Concept designs should be given careful consideration in terms of integration of different modes of transport, circulation, accessibility, way-finding and pedestrian safety. The following factors should be considered during the preparation of alternative designs:

1. Parking requirements.
2. Traffic circulation within the parking zones.
3. Accessibility and connectivity to main roads.
4. Public transportation and taxi stop locations.
5. Dedicated cycle storage areas.
6. Safe pedestrian crossing locations.
7. Taxi drop-off and pick-up locations.

Parking supply should be in accordance with the guidelines on this type of facility.

1.4.2.9 Hospitals

Due to the nature of these facilities, careful planning of access routes and parking facilities is vital for the proper and effective functioning of hospitals. Access for emergency vehicles, which require urgent and safe passage to the facility at any time and in unpredictable volume must be taken into consideration when designing the external road layout.

The Consultant shall consult with the concerned authorities at the outset of the project to clearly establish these requirements.

1.4.2.10 Utilities

Major road and bridge projects typically include improvements to all affected utility services. This also includes preparing plans and specifications for these improvements. Pre-design activities require coordination with many agencies/departments. Final design approval of the utility improvements by the utility agencies is also required. The Utilities Section of the Town Planning Sector is responsible for establishment and approval of all Service Reservations.

Table 1-4 lists the Responsible Agencies/Authorities for Utilities. A survey of existing utilities is required. The purpose of the utilities survey is to determine which utilities can:

- remain in place based on field surveys, as-built plans and other available information;
- need to be replaced/upgraded due to future development;
- be protected and/or relocated; and,
- affect the horizontal and vertical alignment of the roadway.

In the case of future or relocated utilities, it may be necessary to preserve adjacent land for utility installation and relocation. The associated costs for utility work shall be identified as part of the design reflected in the project cost estimate for the Design Concept Report..

Table 1-4: Public Service

Service	Agency/Authority
Water	ADWEA/ADDC (Water) Transco (Water)
Sewer	ADSSC
Telephone	Etisalat / Du
Electricity	ADWEA/ADDC (Power) Transco (Electricity)
Lighting	ADWEA/ADDC (Street Lighting Section) DMT (Lighting Section)
Irrigation	DMT - PRFD
Drainage	DMT – IRID Design Section
Gas	ADNOC
District Cooling	Tabreed
ITS	DMT – Traffic Management Center
Falcon Eye & Security Cameras	National Emergency and Crisis Management Authority Signal Corps Abu Dhabi Traffic Police
Speed and Red Violation Cameras	Abu Dhabi Traffic Police

1.4.2.11 Security

Nearly every project is affected by some level of security issue. All embassies, government installations, palaces, schools, banks and VIP homes are protected by guards with guardhouses, and associated channeling devices. As a result, many of these facilities interfere with road and bridge projects. The DMT *"Abu Dhabi Safety and Security Planning Manual"* is to be followed to ensure adherence to safety and security planning and design principles.

The Consultant is required to minimize the relocation of affected facilities as part of the road and bridge project. As with all other adjacent property improvements, the Consultant is required to provide plans which can be used to construct the necessary improvements either in conjunction with the roadway/bridge project or as a separate project. This is intended so that construction can be undertaken outside of the project right-of-way at the convenience of the affected property owner. Since each case will vary, the limits of improvement, access, facility relocation, parking, etc. requires review by the affected party and DMT. The Consultant is also responsible for assuring DMT that the proposed improvements located outside of the project right-of-way are agreeable to the affected property owner.

1.4.2.12 Commercial Activities

The effects of commercial activities on the road and bridge design shall be taken into account. For example, existing access shall be maintained as well as accommodating special features of the non-project site. As a result, coordination with the Town Planning Sector, adjacent landowners and governmental departments is required to lessen the impact of the road/bridge improvement project on commercial activities.

1.4.2.13 Economics

The Consultant shall assess the economic conditions that exist within the project study area, including income and employment characteristics, tax base and property values. The Consultant shall develop a design that seeks to minimize adverse impacts on these and other economic indicators. This will be done through direct coordination with representatives of DMT.

1.4.2.14 Local Transportation/Circulation

In order to ensure that the project fully incorporates local transportation/circulation needs, the Consultant shall address the following:

- Need for Public Transit Corridors, Stops or Turnouts
- Staging areas for Regional Transportation Hubs
- Police Enforcement Pads
- Pedestrian Walkways and Islands
- Special Landscape Areas

The Consultant should liaise with all relevant parties for the above, including but not limited to, the DMT, DMTTraffic Police and DMT Traffic Services Section.

1.4.2.15 Parking Requirements

In cases where a parking study is required, this should be based on the parking demand calculated based on applicable rates. Reference should be made to the "*Trip Generation and Parking Rates Manual for the Emirate of Abu Dhabi*", published by the DMT.

If a suitable land use code is not specified by the DMT, rates from the Institution of Transportation Engineers may be used, subject to approval of DMT.

Surveys of comparable local developments may also be used, subject to the approval of DMT.

The designer shall provide a table showing calculated parking demand and supply as well as a diagram that clearly shows all parking spaces provided, including provision for disabled people.

The designer shall comply with the requirements of the appropriate agencies in the case of off-street parking facilities and shall ensure that capacity analysis of vehicle access to such car parks, based on the highest peak traffic inflow, is undertaken. Particular consideration needs to be given to access control systems (gates, barriers, ticketing systems, etc.) and their capacity in relation to the expected peak traffic inflow.

The basic dimensions for parking bays are:

- Standard perpendicular and diagonal parking bays Angles 30°, 45°, 60° and 90°: 2.7m (min) x 5.5m.
- Standard parking bay (parallel): 2.7m (2.5m min.) x 6.0m
- Accessible (disabled) parking: 2.5m +1.5m x 5.5m.
- Accessible van parking: 3.35m +1.5m x 5.5m.
- Parking next to walls or physical obstructions: Standard width + 200mm.

Standard parking arrangements are also provided on the DMT Standard Drawings.

1.4.2.16 Recreation

A variety of recreation and leisure activities are available to the residents of Abu Dhabi. These can include, but not limited to, playing fields, parks and streetscape, beach access, clubs, hotels, golf courses, movie theatres and entertainment complexes.

As part of the pre-design activities, the Consultant is required to identify the potential effects on adjacent recreational facilities and minimize the relocation of affected facilities as part of the roadway/bridge project. As with all other adjacent property improvements, the Consultant is required to provide plans which can be used to construct the necessary improvements either in conjunction with the roadway/bridge project or as a separate project.

For design guidance, refer to the latest "*Public Realm Design Manual*" (DMT) and associated PRFD design/review standards from the Parks & Recreation Facilities Division (PRFD).

1.4.2.17 Historical Site Identification and Preservation

The government recognizes the importance of all historical sites and structures that relate to Abu Dhabi's cultural development. The goal of the government is to identify these sites as they are discovered, and where appropriate, preserve the sites.

During the pre-design process, information regarding historical sites shall be compiled from available sources as well as conducting an initial site survey. The Consultant shall also meet with representatives of the Municipality to determine the significance of the site and present recommendations as to appropriate preservation procedures

The Consultant should also liaise with the Environment Agency Abu Dhabi (EAD) for preservation and conservation areas, and the Abu Dhabi Tourism & Culture Authority (TCA) for historical sites.

1.4.2.18 Natural / Environmental Resource Data

DMT regulations require compliance with EAD mandatory procedures relating to environmental considerations. EAD contact details are given below:

EAD - Headquarters

P.O. Box: 45553

Tel: +971 (2) 6934444

E-mail: customerhappiness@ead.gov.ae

Al Mamoura Building (A), Building (62), Al Mamoura St.

Abu Dhabi, United Arab Emirates

Natural/environmental resources within a project study area shall be assessed and considered during development of the project design. The goal is to develop a functional design that avoids or minimizes impact to the natural environment to the greatest extent practicable. To facilitate the planning process involved in the development of the design, the Consultant should map all environmental resources capable of being placed on a map.

The Consultant is to also liaise with EAD.

1.4.2.19 Protection of Existing Amenities

Preservation of any existing landscape treatment or plantation adjacent to proposed roadway projects is extremely important.

PRFD will guide Consultants to the relevant design documents. Pre-design activities include a survey of existing flora and fauna as part of the design survey stage, in accordance with PRFD requirements. The results of this survey are to be agreed with DMT, PRFD and EAD. Road/bridge improvements including utility locations shall be designed to minimize removal of vegetation.

The landscaping survey includes the identification of the number, size, type, condition, and location of all trees, shrubs, succulents, flowers, and grasses. The presence of any vegetation that is specifically protected by decree, or that is considered rare, threatened, or endangered, shall also be identified during the survey. The survey information should then be presented on a scaled plot plan. The scale of each sheet should be adequate to clearly convey the information contained on it. Each sheet should contain a legend, which lists the botanical name of the plant, and its common name. For trees, the size of the tree shall also be listed. Vegetation surveys shall be in accordance with BS 5837:2012.

Public Realm Design

All road schemes in urban areas should consider the full right-of-way corridor (ROW) and therefore include public realm design of the medians, verges and other designated areas within the project limits. The public realm design should be undertaken by the Consultant, ensuring that the following are considered as part of the project, if so required:

- Right-of-way corridor compliance with the latest standards.
  i.e. Urban Street Design Manual (USDM), Utility Corridor Design Manual (UCDM), Public Realm Design Manual (PRDM), Walking and Cycling Master Plan (WCMP).
- Universal access requirements:
  - Pedestrian and cycling connectivity.
  - Furnishings.
  - Planting.
- Public realm lighting.
- Shading.

Water for irrigation should be sourced from the treated sewage effluent (TSE) network, so all large distribution lines require design input and approval from the Abu Dhabi Sewerage Services Company (ADSSC). The Consultant should prepare irrigation designs and obtain PRFD approval and the same for connections to the existing feeder network. Guidance on design submission requirements is available from PRFD. All proposals must conform to the latest PRFD Landscaping and Irrigation requirements to ensure compliance to current standards.

Initial maintenance and operation of the irrigation systems (1 to 2 years) are the responsibility of the Contractor, after which, if agreed to operate and maintain, reverts to PRFD.

PRFD uses DMT third party design review procedures and Standards, mainly concerning the maintenance of all assets (including irrigation network, landscaping, etc.).

1.4.2.20 Topography

Topographic data is important to the development of the Design Concept. Roadway profiles, horizontal alignment, and drainage, are directly affected by topography, which, in turn, affect the project cost. As discussed in Section 201.04, Mapping, the Consultant is expected to review existing maps. In addition, new surveys shall be required to establish the topography for the project.

The Consultant should liaise with the DMT GIS Section for topographical information.

1.4.2.21 Water

The Consultant shall identify and determine the importance of all freshwater and saltwater features within the study area. Aquifers and wells, especially those that supply drinking water, shall also be identified within project limits. In developing the design, the Consultant shall avoid impacts to water resources to the greatest extent possible. If avoidance is not an option, the Consultant shall develop a design that minimizes impact to water resources.

1.4.2.22 Flora and Fauna

The Consultant shall describe any existing wildlife habitat within the project study area. The Consultant is responsible for identifying the types of flora and fauna species, if any, that are likely to utilize the habitat. The Consultant’s design shall avoid, where possible, those habitat areas that support rare, threatened or endangered wildlife species.

1.4.2.23 Air Quality

The Consultant shall assess a project’s affect on existing air quality to determine whether or not it will result in significant deterioration due to increased air emissions.

1.4.2.24 Noise

The Consultant shall assess a proposed project’s affect on ambient noise levels to determine whether or not it will result in a significant deterioration from the existing condition. Noise sensitive receptors, such as mosques, schools and residential dwellings, shall be identified within the project limits. The Consultant shall strive to develop a design that will have the least increase in noise levels to these receptors.

1.4.2.25 Visual / Aesthetic

The Consultant shall assess the existing visual and aesthetic appearance of the project study area. In developing the design, the Consultant should consider the effect that the project will have on the visual and aesthetic environment upon build-out. Views from the project of the surrounding environment as well as views of the project from adjacent vantage points shall be considered. The objective of the design is to develop a project that compliments rather than contrasts the existing visual and aesthetic character of the area.

1.4.2.26 Hazardous Materials

The Consultant shall conduct a survey to identify the actual presence, or likelihood of hazardous material sites within the project study area. Ideally, the project design should be developed to avoid impacting such hazardous sites. This will reduce the health and safety risk and overall project cost. If a hazardous materials site cannot be avoided, the Consultant shall take appropriate steps to remediate the hazardous site prior to construction in order to reduce the potential health/safety risk.

1.4.2.27 Environmental Permit

The Consultant must obtain the Environmental Permit for the concerned project through a third party accredited environmental consultant, in compliance with the relevant regulations of the EAD. The updated list of accredited environmental consultants in Abu Dhabi can be downloaded from the EAD website.

The requirements and processes associated with environmental permitting are described in the *"Standard Operating Procedures for Permitting of Development and Infrastructure Projects in Abu Dhabi"*.

In addition, technical guideline documents are also available including, for example:

- *Technical Guidance Document for Preliminary Environmental Review (PER).*
- *Technical Guidance Document for Environmental Impact Assessment (EIA).*
- *Technical Guidance Document for Strategic Environmental Assessment (SEA).*
- *Technical Guidance Document for Construction Environmental Management Plan (CEMP).*

Other guidelines are available dependent upon the requirements of specific projects and these are available for viewing on the Environment Agency website.

1.4.3 Technical Investigations

1.4.3.1 Introduction

All roadway and bridge projects require technical investigations, to establish the basic building blocks of the design. These technical investigations are initiated in the data collection phase and continue through the development of the Design Concept Report. This subsection identifies the initial activities associated with these investigations. The basic technical investigations include:

- Geotechnical
- Traffic Data Collection
- Survey Control/Field Surveys
- Drainage Surveys.

1.4.3.2 Geotechnical

The objective of highway geotechnical work should be to seek, interpret, and evaluate subsurface and surface data in order to predict the behaviour of the soils and materials along, and adjacent to, the alignment. The resulting information is to be presented in a technical report to be used in the project design.

Data collection includes research of existing geotechnical reports which were prepared for other projects in the geographic area as well as field reviews and preliminary testing. For review of existing geotechnical reports, DMT as well as other Municipality and Government agencies, which hold relevant information of geotechnical information in the immediate vicinity of the project, should be contacted. The existing data will be used to define the number of additional soil borings and the testing requirements for the boring program as per DMT manuals and Specifications. The Consultant shall obtain approval from DMT, Traffic Police and any other concerned agencies prior to commencing geotechnical investigations.

1.4.4 Traffic Data Collection

1.4.4.1 Introduction

Traffic volumes are needed for highway planning, project cost-benefit comparisons, priority determinations, analyzing, monitoring and controlling traffic movement on the highways, traffic accident surveillance, research purposes, highway maintenance, public information, highway legislation and for many other purposes.

However, it should be noted that the traffic data collection and projection techniques described herein are specifically intended for providing traffic volume data required for roadway and bridge design.

The procedures which follow establish the minimum requirements. However, this does not preclude the Engineer from using more sophisticated procedures, including the use of data from permanent automatic collection stations, if available.

The Consultant should coordinate fully with the DMT Traffic Services Section and the DMT.

1.4.4.2 Traffic Projections

DMT roadways are designed to serve traffic volumes anticipated over a 20-year time frame. Projections of future traffic shall primarily be derived from applicable traffic models of the concerned area (obtained from DMT's STEAM model where applicable) or as agreed with DMT.

In cases where traffic model outputs are not required or not available, growth factors derived from historical count data compared with data from recent surveys may be used. The application of such growth factors should be agreed with DMT.

Existing flows may be obtained from existing Automatic Traffic Counters (ATC) located within the city. The Traffic Service Section may be able to provide assistance in this regard.

1.4.4.3 Procedures for Collecting Traffic Volumes

The following sections outline the methods of obtaining traffic volume data.

1.4.4.4 Automatic Traffic Counts

The duration of counts should be agreed with DMT. Data supplied should be on the basis of 24-hour classified counts.

The raw data is to be processed and presented in spreadsheet format and should include:

- Time and date
- Location (coordinates)

- 15 minute and hourly totals
- Totals by vehicle class and direction
- Calculation of morning, afternoon and evening peak hours
- 12, 16 and 24-hour averages for weekdays and weekends (5-day and 7-day averages)
- Graphical summaries as agreed with DMT

Errors and anomalies are to be highlighted and omitted from subsequent analysis.

Unless otherwise specified, ATC data should be classified into the standard 6 classes as listed below or as specified by DMT:

- Motorcycles
- Passenger cars
- Buses
- Heavy Goods Vehicles (HGV)
- Light Goods Vehicles (LGV) and
- Other Vehicles

1.4.4.5 Classified Turning Movement Counts

The time period for the peak hour counts will vary depending on location and project and should therefore be agreed with DMT. However, in the absence of direct guidance, the minimum time periods to capture the peak flows should be:

- AM – 06:30 to 09:00
- Noon – 12:30 to 15:00
- PM – 17:30 to 20:00

The raw data is to be processed and presented in an Excel spreadsheet format to include (at minimum):

- Time and date
- Location (coordinates)
- Schematic plan of permitted movements
- 15-minute and hourly totals by individual movement, by approach, and by vehicle class
- Calculation of morning, afternoon and evening peak hours

For turning count surveys, the preference is for camera recorded data or automatic lane counters where raw data can be retained for verification. When counts are carried out manually, the Consultant should provide evidence of appropriate spot checks carried out by the survey supervisor. At large roundabouts, Automatic Number Plate Matching surveys or Camera Recorded Counts are required, unless otherwise agreed with DMT. In such cases, a detailed methodology of how the counts will be conducted must be set out for approval of DMT.

1.4.4.6 Automatic Speed Surveys

Data should be supplied on the basis of 24-hour counts with the duration of survey in days to be agreed with DMT.

The raw data is to be processed and presented in spreadsheet format and should include:

- Time and date
- Location (coordinates)
- Classification in 10 km/hr groups by direction
- Average and 85th percentile speed in hourly intervals
- 12, 16 and 24-hour average and 85th percentile speeds for weekdays and weekends
- Graphical summaries as agreed with DMT

Errors and anomalies are to be highlighted and omitted from subsequent analysis.

1.4.4.7 Other Surveys

Other surveys that may typically be required include:

- Origin and destination
- ANPR (Automatic Number Plate Recognition)
- Speed radar
- Parking occupancy
- Freight
- Public transportation patronage
- Pedestrian
- Cyclist

In all such cases, a detailed methodology of how the counts will be conducted must be set out for approval of DMT.

1.4.5 Survey Control / Field Surveys

1.4.5.1 Introduction

Each project requires initial field surveys to establish baseline topographic information for project scoping and design. Setting horizontal and vertical control is of great importance in mapping. Relative position in the horizontal plane is maintained by horizontal control. Horizontal control consists of a series of points accurately fixed in position by distance and direction in the horizontal plane.

For most topographic surveying, traverses furnish satisfactory control. For strip maps, the open traverse is used. The open traverse can be tied to fixed points at each end. For area maps, the closed traverse is used. The closed traverse can be closed to form a net which is accurate to the degree required.

Relative position in the vertical plane can be maintained by a series of benchmarks in the map area. These benchmarks are referred to a known datum, usually mean sea level.

1.4.5.2 Horizontal Control

The current inventory of horizontal control points established in the vicinity of the project will need to be investigated. TPS should be consulted on the order of accuracy and status of existing primary and secondary control points.

The need for setting new horizontal control points will be ascertained from the existing data.

1.4.5.3 Vertical Control

The vertical control is to be referred to the Ras Ghumays datum.

Although some authority projects may use their own datum, all DMT design work will be referred to the Ras Ghumays vertical datum.

1.4.5.4 Coordinate System

A Coordinate System has been established by TPS. This system shall be used for all surveys.

1.4.5.5 Field Surveys

Field Surveys will be required on nearly every project to supplement the aerial topography, record underground utility or drainage features, reflect new existing features, provide cross sections and existing pavement elevations at the limits of improvement, obtain building floor elevations and other related information needed for preliminary and final design.

Once the horizontal alignment, including applicable alternative alignments, has been established, the roadway centreline will be staked in the field to enable close examination of the roadway location by DMT representatives and Consultant staff. The staking interval and definition of the project geometrics required will be determined on a project specific basis in consultation with the concerned DMT Municipality Representative.

A detailed survey of the existing greenery impacted by the project will be required. The survey will record the location, size and limits of all trees shrubs and flower beds within the limits of improvement. Photographs should be taken to supplement the data. This information will be recorded on drawings and used to investigate alignment adjustments or alternatives that will minimize the removal of greenery.

1.4.5.6 Drainage Surveys

The Consultant is responsible for a comprehensive survey of drainage facilities and conditions and data collection during the pre-design activities. Reference should be made to the concerned DMT manuals for further details.

1.5 DESIGN CONCEPT REPORT

There are four stages to the design process which are as follows:

- Planning and Study (Pre-Concept).
- Conceptual Design.
- Preliminary Design.
- Final Design.

These are defined in more detail in the DMT *“Consultant Procedure Manual”.*

Note that this section discusses the requirements of the report to be prepared at the conclusion of the concept design stage only and similar reports will be required for the other phases.

1.5.1 Contents

DMT requires the preparation and approval of a Design Concept Report (DCR) prior to commencing final project design. The report is to be prepared under the direction of an experienced engineer designated by DMT. The Design Concept Development details mentioned herein includes a discussion of the background information and data collection activities necessary to develop the design concept. This Section, Design Concept Report, contains a discussion of the specific requirements and content of a DCR.

The role of a DCR is to summarize the needs, alternatives, costs and overall impacts of the proposed roadway or bridge project. The scope of the project is defined and the design criteria identified. The DCR is the project scoping document and the basis for selecting the project design. The basic roadway configurations shown in the DCR will be carried forward to the final design phase.

The preliminary engineering activities associated with the DCR involve preparation of numerous technical studies and reports, many of which are initiated in the data collection phase. These are prepared as standalone documents and are included as Appendices to the DCR. The DCR will summarize the results of these individual reports under the respective topics.

A typical Table of Contents for a DCR is provided in Table 1-5 below, although this may be modified according to the requirements of specific projects.

Table 1-5: Design Concept Report (DCR)

DESIGN CONCEPT REPORT (DCR)
Typical Table of Contents
• Executive Summary • Introduction • Traffic Analysis • Description of Alternatives • Design Data • Typical Sections • Geometrics • Interchange/Intersection Configurations • Road Safety Audit • Parking Study • Hydrology and Hydraulics • Subsurface Investigations and Preliminary Geotechnical Risks and Opportunities Identification • Bridge Type Selection • Utility Impact Analysis • Socio-economic Analysis • Agriculture Impact • Public Feedback • Signing and Pavement Markings • Lighting • Construction Staging • Cost Estimate • Conclusion/Recommendations • Drawings, Plans, Profiles, Typical Sections and Architectural Features

In addition, the Consultant should take into consideration a number of factors when analyzing and designing a project and these general areas are listed below:

Planning and Design.

Transportation Planning, Traffic Engineering and Modelling.

Traffic Impact Studies (including parking study).

Road Safey Audit

Civil and Roads Engineering (related to roads and highways, structures, walls, lighting, road signs and markings, geometrics, etc).

Drainage Engineering (including hydrology and hydraulics).

Structural Engineering.

Geotechnical and Foundation Engineering (including subsurface investigations) and corresponding land use, time and cost decisions.

Topographical and Bathymetric Surveying.

Architecture (related to any proposed structures).

Landscape Architecture.

Pedestrian Movement.

Electromechanical, Utility and Lighting Engineering (including utility impact analysis).

Environmental Considerations.

Economic Feasibility Studies (including socio-economic analysis).

Operation and Management Strategy.

Stakeholder Requirements.

Quantity Surveying (including cost planning and life cycle costing).

Constructability.

Value Engineering.

Risk Management.

Safety.

Sustainability.

Project Schedule.

Completed Checklist as per QA/QC Procedures.

Furthermore, the discussion under each topic will address interdisciplinary relationships necessary to coordinate all technical aspects of the design concept. The sections that follow provide guidance for the development of the technical studies and requirements for presentation of the material in the DCR.

1.5.1.1 Format

The DCR will be prepared and packaged as follows:

DCR (Volume I).

Written portion of the report bound separately in A4 size.

DCR (Volume II).

Drawings that accompany the report bound separately in A3 size.

DCR (Appendices).

Technical Memorandums, Studies and Reports bound in A4 size. For smaller projects the documents should be bound together. Larger projects may require separate packaging of the reports, titled as Appendix A, Appendix B, etc.

Each document will include the following information on the cover:

DMT and/or concerned Municipality logo.

Project Title.

Project Location Plan.

Design Concept Report.

Volume No. or Appendix No.

Project Number.

Date.

Client and department details (i.e. Internal Roads and Infrastructure (IRI) Directorate).

Consultant details.

See Figure below, which is to be used as the standard cover sheet for the DCR. As mentioned, the logo has to be as per the concerned governing authority.

بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

Project Title: ………………………………………………………………

………………………………………………………………

Project Location Plan

Final Design Concept Report

Volume I

Project No.: …………

Date: …………

Preparer's

Prepared by: …………

Address: …………

Client

Prepared for: …………

Section: …………

Figure 1-4: Standard Design Concept Report Cover Sheet

1.5.2 Executive Summary

The Executive Summary is a short (2-4 pages) recapitulation of the DCR document. The Summary should address the following key topics:

Purpose and Need of the Project.

Alternatives Evaluated.

Recommended Design Concept.

Major or Controversial Issues.

Estimated Cost.

Conclusion.

It is not necessary to address every aspect or technical consideration that is discussed in the main body of the report. The summary should focus on items presented in the report that are of critical interest to DMT such as an accurate concise description of the recommended design concept and the estimated cost. It should be clearly stated how the recommended design responds to the purpose and need of the project. Both the major benefits (e.g. improved traffic circulation, improved intersection safety) and the adverse impacts (e.g. displacement of coastal vegetation) should be summarized.

1.5.3 Introduction

The introduction is to prepare the reader for the subject matter that will follow in the body of the report. It should only be a few paragraphs in length and should provide a brief description of the project as well as the reason for preparing the Design Concept Report. The project description should be very general and should identify the project's location, the agency/municipality in charge of its implementation, and the source of funding that will be used for its design and construction. A statement can also be included that identifies how the project fits into the overall transportation infrastructure of the area.

1.5.4 Traffic Analysis

All highway projects are subject to the approval of the DMT in terms of the traffic implications of the scheme. As part of this process, project information is to be submitted to the DMT as an initial application, which will be used to establish whether a Transportation Impact Study (TIS) is required and if so, the required extent of the study area. This will also define the requirements for traffic counts.

The collection of traffic data and the traffic projection procedures are as per DMT procedures and manuals such as the Transportation Impact Study Manual. The data will be used to analyze and shape the various alternatives and geometrics. This is an iterative process that results in identification of the number of through lanes, auxiliary lanes, turning lane requirements including storage lengths, signal warrants, level of service and capacity. Schematic diagrams of the roadway segments and intersections should be used to display the data. This information will be presented in the DCR along with a summary of the project traffic data including current and forecast ADT values, peak hour and peak hour directional splits and percentage of trucks.

Traffic signal recommendations will be included in the report. For each signal location, the following information should be provided:

Phasing Diagram.

Controller Equipment.

Detection Requirements.

CCTV.

Interconnection.

Power Source.

Note that traffic signals are the responsibility of the DMT and as such these proposals require the review and approval from DMT.

On all projects where the primary justification, or an important justification, of the project is to improve safety, the DCR should include accident history data and an analysis of the causes of the accidents as well as a collision diagram. Estimates should be made of the accident reductions expected if the improvement proposal (or alternatives) is built. The monetary value of the accident savings should be calculated over the design period of the project (normally 20 years where geometric improvements are proposed).

A summary of the traffic analysis shall be included in the body of the DCR. The complete report shall also be included as a separate Appendix.

1.5.5 Description Of Alternatives

In consultation with DMT, the engineer shall develop alternatives to be evaluated that respond to the project purpose and need to varying degrees. The alternatives identified may include separate horizontal alignments, profile variations, typical section concepts, etc., that can be evaluated in a matrix form to qualitatively and quantitatively review the alternatives to identify major differences. The engineering, social, economic and natural environmental impacts for each alternative under consideration must be addressed.

The horizontal alternative alignments will be displayed on aerial photographs for evaluation of associated impacts. The sheets will show the proposed centreline, stationing, proposed structures, edge of pavement lines, utilities and affected properties, at a scale that is appropriate to the project length and character.

A cost estimate will be prepared for each alternative and include:

Construction costs.

Utility relocation works costs.

Land acquisition costs.

At this point, meetings will be held with various Municipality and Government Departments that have a vested interest in the project. The engineer will present the alternatives, review the evaluation criteria and matrix form and discuss merits and adversities of the different alternatives. Comments and direction received at the meeting(s) will be factored into the alternatives evaluation matrix.

Finally, the analysis will conclude with a discussion of the evaluation criteria for each matrix parameter, input/direction received concerning the project and a summary discussion of the advantages and disadvantages of each alternative studied. This will be followed by the engineer's recommended alternative with supporting justification for the selection.

1.5.6 Design Data

This section will document the design criteria associated with the recommended design concept and specifically identify any exceptions from the minimum criteria established for the roadway classification.

It is very important that sufficient detail is included in the DCR so that future revisions to basic design features and project scope are kept to a minimum.

The following basic design criteria shall be included:

the functional classification of the road;

the *minimum* design speed(s), minimum horizontal/vertical curve radii, minimum sight distances (passing and stopping), maximum superelevation and other design requirements associated with the classification of the road;

the *actual* design speed(s), horizontal/vertical curve radii, sight distances (passing and stopping), superelevation, etc., used for the project;

lane width, shoulder width and bridge width on the project;

cross slope and grade;

horizontal and vertical alignment (actual);

horizontal and vertical clearance; and,

bridge structural capacity.

The design exceptions identified shall be prepared in a “Fact Sheet” format as described in Chapter 2 of this manual.

1.5.7 Typical Sections

The typical roadway cross sections and the dimensions of the lanes, shoulders, median(s) for both the mainline and all ramps are to be identified. The number of typical sections will depend on the number of significantly different roadway/pavement structure conditions. At a minimum, at least one section should be provided which depicts all facilities within the limits of the right-of-way (i.e., ramps, frontage roads, drainage channels, etc.).

The type of roadway section, i.e., cut or fill, number of lanes, shoulders, pavement structural section, cross slopes, and any retaining walls are also to be included. Drawings that illustrate this information are to be included in the Appendix to the DCR.

1.5.8 Geometrics

The alignment, profile, and number of traffic lanes, including through lanes, auxiliary lanes, turning lanes and ramp lanes are to be plotted on an appropriately scaled plan. A scale of 1:1,000 should be used for urban projects, unless otherwise agreed with DMT. The alignment should be displayed on an aerial base and the corresponding roadway profile shown below in a split sheet format.

The text in this section should include a narrative description of the geometrics, constraints, controlling factors, drainage considerations and reference to the design exceptions. The plans are to be attached as an appendix to the DCR.

1.5.9 Interchange / Intersection Configuration

The various types of traffic interchanges are described in Chapter 12, Interchanges. The discussion in this section should identify the *site* and *project* considerations which led to the selection of the interchange and intersection type.

The *site* considerations include:

Constraints imposed by the existing and nearby transportation facilities.

Proximity of adjacent interchanges.

The standards and arrangement of the local street system including traffic control devices.

Right of way controls.

Local planning.

Community impact, and cost topography.

The *project* considerations include:

Speed, volume, and composition of traffic to be served.

The number of intersecting legs.

Crossing and turning conflicts.

Safety considerations, including Road Safety Audits requirements.

Cost.

The interchange/intersection alternatives should be evaluated as a part of the alternatives analysis when viable options are identified for the particular project. This is especially true for freeway and urban expressway projects where the Interchange/Intersection type has a significant impact on the project character, capacity and cost.

1.5.10 Parking Study

In accordance with Parking Requirements, a parking study shall be prepared and included as part of the DCR.

The results of the study shall be summarized in the body of the DCR, with the entire study included in the Appendix.

The summary of the results shall include:

Existing parking demand.

Anticipated parking demand.

Resulting parking shortfall (or excess).

Alternatives as to how the project can provide adequate parking.

Cost comparison of parking alternatives.

Economic impact of inadequate parking.

If required by the roadway classification, the need for off-street parking facilities.

Costs and right-of-way requirements associated with each of the above alternatives.

Recommended alternative to meet the anticipated parking demand.

Conceptual design of the recommended alternative.

Note that the requirements for parking should be in accordance with the latest guidelines published by the DMT and that all parking proposals shall be approved by the DMT.

1.5.11 Hydrology And Hydraulics

The requirements associated with hydrology and hydraulics can be found in the concerned DMT manuals.

The Design Concept Report shall include a separate section (study) for drainage design concepts, which shall also include, when required, separate reports for flood plain encroachment and major waterway crossing studies.

The drainage design concepts section shall address the following items:

- Planning consideration for the overall watershed considering the project and other existing and future development.
- Assessment of existing and future conditions affecting drainage areas, flow patterns, and flood levels.
- Estimate of future development and its effect on flows and flood levels.
- Drainage map showing topographic features, watershed boundary, slope contours, drainage areas, existing drainage systems, proposed cross-drain locations (including peak flow volume, design high water elevation and culvert size) and proposed conveyance systems (pipes and channels including flow direction, sizes and peak flow volume)
- Hydrology calculations for drainage area intercepted by the project to include peak runoff volume flow rates from each drainage area.
- Proposed concepts for disposal of storm water.
- Design criteria, procedures, methodology, and assumptions for analysis and design.
- Proposed concepts for handling and disposing of storm water during construction.
- Recommended size and location of cross drainage structures and channels, including design high water elevation that might affect the road profile grades or the roadway location.
- Proposed concepts for on-site roadway drainage collection, detention, and outfall locations.
- Separate Flood Plain Study Report where the roadway encroaches on flood plains either longitudinally or transversely.
- Bridge Location and Hydraulics Report for bridge or large box culvert waterway crossings.

1.5.12 Subsurface Investigations

Once the project location, horizontal and vertical alignment and structure requirements have been generally defined, the engineer will formulate a subsurface exploration and testing program. The objective of the exploration program is to provide specific subsurface information along successive design sections or reaches of the project. The data will allow some basic judgments to be made, i.e.

the most suitable type(s) of foundations for structures and recommended pavement designs to be developed during the design phase.

In the case of either the structure borings or roadway borings, the geotechnical program will serve to reveal the type, severity and extent of geotechnical design problems.

The Geotechnical Report will assemble the results of the subsurface exploration program, analyze, and make geotechnical engineering recommendations using the field boring and lab test data. This will be presented in an engineering report, prepared by the engineer for the project and included in the Appendix. The results will be summarized in the DCR.

The report is to contain the following information:

Summary of previous geotechnical investigations.

Description of the program undertaken to identify geotechnical and subsurface elements which affect project design.

Results of surface visual observations.

Groundwater data.

A summary of the information obtained from and the location diagram of the soil borings.

The general description of the subsurface geological strata obtained from the soil borings, including any areas of unacceptable soil conditions.

Particle size analysis and potential for scour.

Results of any material testing.

Analysis and recommendations for embankment construction including settlement and surcharging.

Analysis and preliminary recommendations for pavement structural section and foundations.

Physical and chemical soil stability testing and analysis.

Preliminary assessment of safe slopes.

Selection of suitable foundation system alternatives.

Preliminary and simplified analysis and computations – may need to be carried out to arrive at initial conclusions on selection of foundations and slope systems.

Suggest ground improvement method and suitable alternatives.

1.5.13 Bridge Type Selection

Selection of the most suitable type of structure involves investigating alternative superstructure, substructure and foundation types including variation of span length and structure depth to determine the best bridge type and arrangement for a particular site. This is an iterative phase where assumptions must be made and later verified or modified during the design process. Detailed design should not be performed at this stage unless it is necessary to confirm the adequacy of a concept. When performing the concept studies the following design objectives shall be considered as a minimum:

Safety

Serviceability

- Durability
- Inspectability
- Maintainability
- Rideability
- Utilities
- Deformations
- Consideration of Future Widening

Constructability

Economy

Bridge Aesthetics

Plans and sketches should be made of the various alternatives investigated and included in the report.

Both the vertical and horizontal clearances should be checked to ensure that adequate clearances are provided. Inadequate vertical clearance will necessitate a change in either profile grade or superstructure depth while inadequate horizontal clearance may necessitate a change in span length.

The geotechnical aspects of the site should be considered since the foundation type and associated cost may influence the type of bridge selected. An initial (stage one) subsurface exploration and testing program will be performed in parallel and will be used to determine foundation type and costs.

Traffic requirements must be investigated including any detours or phasing requirements.

The above guidelines describe the process during the conceptual design stage. The requirements for other stages are detailed in the DMT *"Consultant Procedure Manual"*, which also includes the Project Submission Requirements Form.

1.5.13.1 Bridges Over Waterways

For waterway crossings, coordination with the project drainage requirements will be necessary. The designer should obtain the Initial Drainage Report and thoroughly review the contents before starting the analysis of alternatives. For navigable crossings, the channel width, vertical clearance, pier protection and navigational aids should be investigated and agreed.

1.5.13.2 Widenings / Rehabilitation

On projects involving widening, in addition to the requirements for new bridges, the following items should be investigated:

The existing structure should be checked for structural adequacy.

The condition of the existing deck joints

The condition of the existing bearings.

The condition of existing diaphragms on steel girder bridges.

The existing foundations.

The existing waterway opening, vertical and horizontal clearances.

The need for adding approach slabs.

The adequacy of existing bridge rail.

When the above items have been investigated, concept design can proceed by studying alternatives. Possible alternatives include: widening to one side, widening symmetrically on both sides or replacing the bridge with a new structure. Approximate costs based on preliminary quantities and unit costs associated with each solution will be required.

1.5.13.3 Bridges and Highway Structures Concept Report

The Bridges and Highway Structures Concept Report is prepared in the concept design phase, describing the structural design options considered and summarizing the findings of the geotechnical and initial drainage studies. Generally three or more alternative conceptual structural solutions should be assessed, based on the criteria listed in the applicable DMT manuals. Ultimately, one of these options should be recommended for DMT approval.

For large or sensitive projects, approval by the Executive Council or higher authority will also be required. These may be individual or joint discussions as dictated by the size, location, complexity, and sociological, economical, ecological and environmental demands of the project.

Through these discussions a structure with architectural features that are compatible with structural, safety and site requirements can be developed.

The completed Bridges and Highway Structures Concept Report shall include general conceptual plans of the bridges proposed on the project. These plans reflect the bridge geometrics, architectural themes, substructures and types of foundations. A complete discussion of the costs and feasibility of alternate designs must be included. This is especially important for unusual and major structures.

The results of the bridge type selection process will be summarized in the DCR, and should thoroughly discuss the factors that influenced the selection of the preferred alternative. This should include the following structural considerations:

Foundation Type

Substructure

Superstructure

Architectural Features

Vertical and Horizontal Clearance

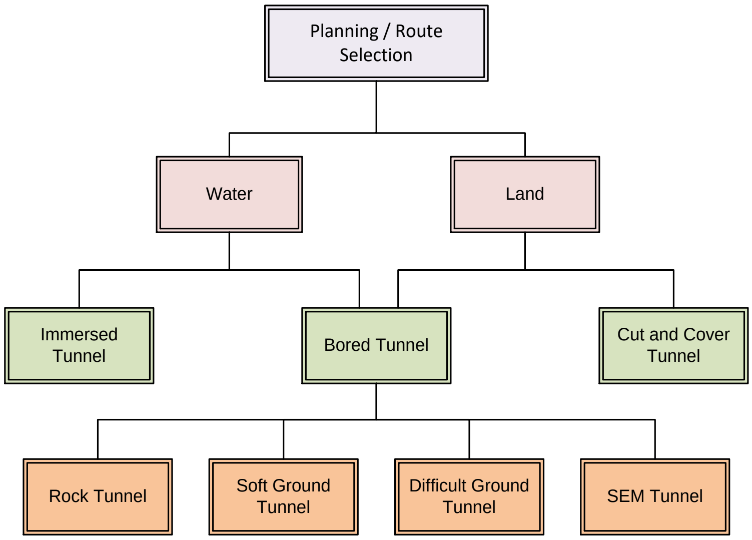
Other Key Factors (as specified in the relevant checklist in the DMT *"Quality Control and Quality Assurance Procedures"* manual).

The General Plan shall be included in the Drawings (A3 size) that will accompany the DCR.

1.5.14 Tunnel Selection Criteria

There are a number of principal types and methods of tunnel construction. These are cut-and-cover, bored or mined tunnels, rock tunnels, soft ground tunnels, immersed tunnels and jacked box tunnels. The "*Technical Manual for Design and Construction of Road Tunnels - Civil Elements",* US Department of Transportation Federal Highway Administration, Publication No. FHWA-NHI-10-034, December 2009 provides details and uses of the different type of tunnels.

The preliminary choice of tunnel type, during the conceptual design stage, can be dictated by the general ground conditions, as illustrated in Figure 1-5 below:



SEM : Sequential Excavation Method

Figure 1-5: Preliminary Road Tunnel Type Selection Process

The final selection of a tunnel type depends on the geometrical configurations, the ground conditions, depth of cover to the tunnel, the type of crossing, the anticipated ground movements during construction and environmental requirements. Therefore, it is important to perform the tunnel type study as early as possible in the planning process and select the most suitable tunnel type for the particular project requirements.

Determining the suitable type of tunnel at selected locations will depend on, but not be limited to, the following:

- Defining the functional requirements, including design life and durability requirements.
- Evaluation of available investigations including geological, geotechnical and geo-hydrological data.
- Evaluation of the environmental, cultural and institutional studies.
- Tunnel alignment, profile and cross section.
- Outcome of risk analysis and associated mitigation measures to identify construction and long term risks.

Tunnel alternatives shall also be included in the Bridges and Highway Structures Concept Report. This should include information used to confirm tunnel options considered, available and additional investigations, results of geotechnical assessment, risk and environmental assessments and recommendations. The assessment should include conceptual structural arrangements.

The choice of tunnel type requires the approval of DMT. The level of necessary approvals will be dictated by the size, location, complexity, sociological, economical, ecological and environmental demands of the project.

The Consultant should take additional care to ensure that proposed tunnels do not impact negatively on existing structures in the vicinity of the project. Additional analysis should be conducted to ensure that any proximity or interface with such structures does not impart any detrimental effect.

1.5.15 Utility Impact Analysis

Utility impacts are a key project issue, especially within existing transportation corridors. Data collection and coordination with the various agencies/departments is required. The second phase of work includes analysis of the existing and proposed utilities with respect to each alternative in order to permit estimation of costs and evaluation within the alternatives matrix.

Utility corridors including proposed Service Reservations should be identified and indicated on the typical sections and roadway plans included in the DCR. For urban projects, the location of service reservations will affect the roadway geometrics including parking areas, green areas and the proposed pavement surfacing.

The DCR will include a thorough discussion of the utility impacts and a tabulation of the existing utility inventory as follows:

Item Number

Owner

Description

Station

Location

Status

Remarks

The DCR will summarize the impacts for each major utility (water, sewer, telephone, irrigation, electrical, etc). The responsibility for design and construction of the facilities will be addressed. Schematic plans showing the major existing and proposed utilities should be prepared and included in the drawings section. Recommendations will be given for general utility relocation schemes and for resolution of specific utility conflicts. Associated utility costs will be included in the concept cost estimate.

For larger projects a separate Utility Report should be prepared and included as an Appendix to the DCR.

Reference shall also be made to the latest version of the DMT "*Utility Corridors Design Manual*" (UCDM) for further guidance.

1.5.16 Socio - Economic Analysis

An analysis and discussion of the socio economic data as per the DMT Feasibility Study Guidelines shall be included in the DCR. Each of the topics covered in the abovementioned Guidelines should be included or, if not relevant, it should be so stated including the reason why it is not relevant.

For any of the topics which are *not* relevant, prior approval from DMT is required to exclude the issue from the DCR. The required information as to the reasons why topics are not relevant shall be summarized in a concise Technical Memorandum accompanied by supporting documentation as necessary. DMT shall make a determination as to the relevance of the topic based on this information. The Technical Memorandum and supporting documentation is to be included as a separate appendix in the DCR.

The above information may also be obtained from the Environmental Impact Assessment Report, where applicable.

1.5.17 Agriculture Impact

Agricultural resources are important to man's survival and therefore must be preserved to the greatest extent possible. The Consultant shall identify the potential impact that the proposed project alternatives may have on these resources within the study area. Primarily, this involves determining whether or not the project will directly impact (i.e. irreversibly commit) land that is presently used for agricultural purposes. In the description of impact, the Consultant shall identify whether the land is actively farmed or fallow as well as the types of crops that would be affected. Impacts will be quantified in hectares. Indirect impacts will also be identified and described. These may include, but are not limited to, the potential disruption of the existing irrigation system or pollution of nearby agricultural lands from untreated stormwater runoff. Impacts associated with each project alternative will be compared and the alternative with least agricultural impact shall be identified if such an alternative exists.

The above information may also be obtained from the Environmental Impact Assessment Report, where applicable.

1.5.18 Public Feedback

Public involvement is an important aspect in the overall success of a project. At the onset of the project, the consultant shall develop a Public Involvement Plan that will establish the approach to be used to coordinate project planning and details with the public. In addition to keeping the public informed of the project, the plan will also provide the public with the opportunity to comment at various stages of project development. By soliciting and actively considering public input, the Consultant is more likely to produce a design that is economically feasible and acceptable to the public.

This section of the DCR should briefly describe the elements of the Public Information Plan, including the location and scheduling of public information meetings, workshops and consensus building sessions, or any other forums aimed at soliciting public input. A summary of the primary issues raised by the public should be presented along with a discussion of how these issues have been addressed during the development of the project, and whether or not consensus has been reached. A file should be maintained as backup for each public meeting that contains a list of participants and the issues raised.

1.5.19 Signing and Pavement Markings

Signing concept plans will be developed to show the major guide signs required for the proposed facility in accordance with the **DMT *"Manual on Uniform Traffic Control Devices (MUTCD)"***. It may be necessary to include signing outside of the project limits. New signs or modifications required to existing signs shall be clearly identified. The signing requirements shall be displayed on a reduced scale version of the project geometrics sufficient to show the required detail. Proposed guide signs should be illustrated graphically with arrows pointing to the sign location. Signing requirements associated with the construction staging/detour scheme should also be discussed.

The signing and lighting concept plans will be included in the drawings section of the DCR.

1.5.20 Lighting Concepts

This section should begin with a discussion of the design criteria that governs the location of lighting, the type of lighting relevant to the roadway classification or route and the method of illumination analysis. Applicability or conformance to existing Master Lighting Plans must be considered. Alternative types of lighting such as high mast at major interchanges should also be addressed. The typical spacing between light sources, and the compatibility with adjacent or intersecting lighting system will be shown and illustrated on schematic plans.

1.5.21 Construction Staging

Maintenance of traffic during construction can have a significant effect on the surrounding traffic system, in terms of public convenience, design, cost and the duration of construction. The DCR shall include a discussion as to how construction of the project will be staged including:

Number of Stages

Erection of Falsework

Anticipated Detours

Duration of each Stage

The final design plans will generally be prepared in conformance with staging described in the DCR. In accordance with the *"Urban Work Zone Traffic Management Manual"*, traffic management plans, work zone practices, markings and signing are to be submitted at the relevant design stages and pre-construction stage to the concerned Municipality for review and approval.

1.5.22 Cost Estimate

The DCR concept cost estimate must be as realistic and accurate as possible. The degree of effort and detail for each project is expected to vary depending upon the complexity and sensitivity of the project-related issues.

The concept cost estimate should be prepared using the "Concept Project Cost Estimate" form as shown in Table 1-6 to summarize the individual bills. This is intended to standardize the format and type of items that need to be considered in the project, consistent with the Standard Specifications. Similar forms must be developed for each bill section to back-up the summary, including the estimated quantities and unit prices. It is important that all known items of work be identified and estimated. In some instances, not all of the items can be identified at this stage and an appropriate contingency factor should be applied to reflect possible increases such as modification of the project limits or adding decorative features.

Table 1.5: Cost Estimate Worksheet
Concept project cost estimate
Summary of bills of quantities

BILL NO.	BILL DESCRIPTION	AMOUNT IN FIGURES	
		Dh	Fs
I	GENERAL		
II	EARTHWORKS		
III	SUBBASE AND BASE COURSES		
IV	ASPHALT WORKS		
V	CONCRETE WORKS		
VI	STORM WATER DRAINAGE SYSTEM		
VII	WATER WORKS		
VIII	PRESTRESSED CONCRETE WORKS		
IX	TRAFFIC MARKINGS AND SIGNS		
X	SITE LABORATORY		
XI	CONCRETE PILE FOUNDATIONS		
XII	METAL WORKS		
XIII	POST-TENSIONED CONCRETE		
XIV	EXPANSION AND FIXED JOINTS		
XV	IRRIGATION WORKS		
XVI	LIGHTING AND ELECTRICAL DISTRIBUTION WORKS		
XVII	TRAFFIC CONTROL SYSTEM		
XVIII	DAILY WORKS SCHEDULE		
XIX	TELEPHONE WORKS		
XX	SEWERAGE WORKS		
XXI	STREET FURNITURE		
XXII	SOFT LANDSCAPING		
XXIII	HARD LANDSCAPING		
XXIV	ARCHITECTURAL WORKS		
	TOTAL ESTIMATED COST		

Note: Enabling works to be included in Bill No. II – Earthworks.

1.5.23 Conclusions / Recommendations

This section will include conclusions, recommendations, and their associated costs. The name and title of the Project Engineer responsible for the preparation of the DCR as well as the DMT Engineer who served as the DMT Representative shall also be indicated.

1.5.24 Appendix

This section will be used for appending Technical Memorandums and the complete detailed studies or reports as directed by DMT. Examples of these are as follows:

Utilities Report.
Traffic Analysis Report.
Road Safety Audit Report
Lighting Report.
Bridges and Highway Structures Report.
Pavement Design Report.
Initial Drainage Report.
Geotechnical Report.

1.5.25 Drawings

The drawings prepared to illustrate and define the design concept should be presented in A3 format as Volume II of the written report which is bound separately in A4 format. The drawings should include the following:

Typical Sections.

Alternatives.

Structure/Bridge/Tunnel General Plans.

Roadway Plan/Profile.

Signing and Lighting Concept Plans.

Utilities.

Drainage.

Architectural Renderings.

Construction Staging Schematics.

Other project specific plans as require.

Fact Sheet - Design Exceptions.

Parking Study.

2 GENERAL DESIGN CRITERIA AND CONTROLS

2.1 Overview

A range of factors influence design choices for road projects. Design characteristics and values adopted must provide a satisfactory service to roads users, and be economically viable within the financial, topographical, and environmental constraints that may exist.

There are many aspects to be considered in the planning and design of road projects. Chapter 2 provides guidance on:

- context sensitive solutions;
- sustainability;
- functional classification;
- speed (design speed, average running speed, operating speed);
- design vehicle characteristics;
- traffic volume controls and composition;
- environmental considerations;
- right-of-way considerations;
- access control and access management; and
- non-road design controls (e.g. drivers, pedestrians, cyclists).
- safety

Other aspects that are also to be considered include land use, topography, and geotechnical considerations.

2.2 Context Sensitive Solutions

Context sensitive solutions (CSS) is a collaborative, interdisciplinary approach that involves all stakeholders to develop a transportation facility that fits its physical setting and preserves scenic, aesthetic, historic and environmental resources, while maintaining safety and mobility. CSS is an approach that considers the total context within which a transportation improvement project will exist.

A design for new road or major reconstruction should take into account the following criteria:

- safe access for other modes of transportation (e.g., pedestrians, transit, cyclists);
- the constructed and natural environment of the area; and
- the environmental, scenic, aesthetic, historic, community, and preservation impacts of the activity.

2.2.1 Basic Principles

The following are the basic principles behind context sensitive design:

1. <u>Qualities of Excellence in Transportation Design</u>. The following apply:

   - The project satisfies the purpose and needs, as agreed to by a full range of stakeholders. This agreement is forged in the earliest phase of the project and amended as warranted as the project develops.

- The project is a safe facility for both the user and the community.
- The project is in harmony with the community, and it preserves environmental, scenic, aesthetic, historic and natural resource values of the area (i.e. exhibits context sensitive design).
- The project meets or exceeds the expectations of both designers and stakeholders, and achieves a level of excellence in people’s minds.
- The project involves the efficient and effective use of the resources (e.g. time, budget, community) of all involved parties.
- The project is designed and built with minimal disruption to the community.
- The project is seen as having added lasting value to the community.

2. <u>Characteristics of the Process Contributing to Excellence</u>. The following apply:

- Communication with all stakeholders is open, honest, early, and continuous.
- A multidisciplinary team is established early, with disciplines based on the needs of the specific project and with the inclusion of the public.
- A full range of stakeholders is involved with transportation officials in the scoping phase. The purposes of the project are clearly defined, and consensus on the scope is forged before proceeding.
- The roadway development process is tailored to meet the circumstances. This process should examine multiple alternatives that will result in a consensus of approach methods.
- A commitment to the process from top agency officials and local leaders is secured.
- The public involvement process, which includes informal meetings, is tailored to the project.
- The landscape, the community, and valued resources are understood before engineering design is started.
- A full range of tools for communication on project alternatives is used (e.g. visualization).

The principles of context sensitive design/context sensitive solutions apply essentially to any transportation project with the aim being to ensure that the full range of stakeholder values is incorporated into the design process and final result. The *Road Geometric Design Manual* prescribes typical design practices, but also recognises that some flexibility may be required where the design criteria may be inappropriate.

2.2.2 Roadway Design

The roadway consists of a number of different elements that, when combined together, provide for a safe and efficient facility. Geometric design must weigh the needs of the facility from a mobility, safety, and access point of view, along with land use, environment, community needs, values, aesthetics, and likely users.

Within the various contexts, the considerations and trade-offs associated with different features vary. For example, the speed of the facility will have a high impact on the elements that are appropriate for the facility (e.g. high-speed freeways require wider lanes; while narrow lanes are acceptable on low-speed urban streets). The mix and types of users are other elements that will directly affect the needed and appropriate elements of the transportation facility through the project area.

Identifying opportunities to maximise the mobility and safety for all stakeholders is a primary challenge in the project development. This requires the designer and project stakeholders to understand where there is flexibility in the project criteria and to assess the potential impact for revising the criteria.

It is essential that the designer understanding the functional basis for each design element, not just what are the minimum design values. Design criteria generally reflect a safety conservative philosophy within the context of overall cost-effectiveness. Acceptable design values for any feature are established to ensure that the feature itself will not measurably contribute to an increased risk of a crash. Designers may occasionally accept a design dimension outside the customary range (e.g. lane widths) to provide an acceptable solution. Table 2-1 identifies potential impacts that may occur when changing design speed, lane widths, and shoulder widths and where all other features are held constant. Using design dimensions outside of the customary range may require mitigating any negative impacts with other measures (e.g. providing additional clear zone for sharp horizontal curves). The designer should avoid changes that simultaneously combine multiple dimensions outside traditional ranges. Multiple impacts may counter act each other and/or cause additional impacts.

2.2.3 Additional Resources

For additional guidance on context sensitive solutions, the user should review the following documents:

- Abu Dhabi Urban Planning Council *Abu Dhabi Urban Street Design Manual* (4)
- AASHTO *A Guide for Achieving Flexibility in Highway Design* (5),
- US Federal Highway Administration’s *Mitigation Strategies for Design Exceptions* (6),
- *NCHRP Report 480 A Guide to Best Practice for Achieving Context Sensitive Solutions* (7)
- *An ITE Recommended Practice, Designing Walkable Urban Thoroughfares: A Context Sensitive Approach* (8)
- *NCHRP Report 642 Quantifying the Benefits of Context Sensitive Solutions* (9)
- *NCHRP Synthesis 422 Trade-Off Considerations in Highway Geometric Design* (10)
- *Understanding Flexibility in Transportation Design – Washington* (11)

Table 2-1: Potential Impacts from Changes in Design Criteria

Feature	Change	Potential Impacts
Design Speed	Increase	• Shorter travel times (depends on LOS) • Reduced opportunity to view features and services adjacent to roadway • Decrease in safety
	Decrease	• Increased opportunity to view features and services adjacent to roadway • Improved pedestrian/bicyclists environment • Increase in safety
Lane Width	Increase	• Additional room for vehicles to manoeuvre • Higher operating speeds • Increased impervious surface • Increased capacity • Longer pedestrian crossing distances — greater risk • Can provide room for turning movements at intersections • Can provide room for additional lanes • More room for bicyclists
	Decrease	• Reduced room for vehicles to manoeuvre • Reduced capacity • Reduced vehicle speeds • Shorter pedestrian crossing distances • Increase in safety for pedestrians
Shoulder Width	Increase	• Increased space for errant and disabled vehicles • Increased impervious surface • Increased impervious area to be mitigated • Longer pedestrian crossing distances
	Decrease	• Reduced area for errant or disabled vehicles • Reduced impervious area to be mitigated • Shorter pedestrian crossing distances

Source: (11)

2.3 Sustainability

The overall goals of sustainable development are to meet human need and improve quality of life to live within the earth's ecological carrying capacity, and maintain or enhance its natural capital; and to protect future generations from reduced quality of life. These goals are achieved by addressing three dimensions of sustainability:

1. Economic Development. Ensure that the financial and economic needs of current and future generations are met.

2. Environmental Stewardship. Ensure a clean environment for current and future generations and use resources sparingly.

3. Social Equity. Improve the quality of life for all people and promote equity between societies, groups, and generations.

Estidama, which is the Arabic word for sustainability, is the intellectual legacy of the late Sheikh Zayed bin Sultan Al Nahyan and a manifestation of visionary governance promoting thoughtful and responsible development. The leadership of Abu Dhabi are progressing the principles and imperatives for sustainable development, through Estidama, while recognising that the unique cultural, climatic, and economic development needs of the region require a more localised definition of sustainability.

The ultimate goal of Estidama is to preserve and enrich Abu Dhabi's physical and cultural identity, while creating an always improving quality of life for its residents on four equal pillars of sustainability: environmental, economic, social, and cultural. For additional information on Estidama in Abu Dhabi, see www.estidama.org (12).

Transportation plays a key role in the global economy, and in the challenges and opportunities associated with sustainable development. Sustainable transportation can be viewed as an expression of sustainable development in the transportation sector. Sustainable transportation addresses local, regional, national, and global issues and, therefore, requires considerable coordination. It is important to apply sustainable transportation in a holistic and integrated manner across the various sectors (external to transportation) to ensure that key concerns such as depletion of resources, global climate change, disruption of ecosystems, and toxic pollution are effectively addressed.

A comprehensive definition of a sustainable transportation system states that sustainable transportation:

- allows the basic access needs of individuals and societies to be met safely and in a manner consistent with human and ecosystem health, and with equity within and between generations;
- is affordable, operates efficiently, offers choice of transport mode, and supports a vibrant economy; and
- limits emissions and waste within the planet's ability to absorb them, minimises consumption of non-renewable resources, limits consumption of renewable resources to the sustainable yield level, reuses and recycles its components, and minimises the use of land and the production of noise.

For additional information on sustainability, the user should review the following documents:

- Abu Dhabi Urban Planning Council *Abu Dhabi Urban Street Design Manual* (4)
- Abu Dhabi Urban Planning Council *Public Realm Design Manual* (13)
- US Federal Highway Administration's *Transportation Planning for Sustainability Guidebook* (14)
- "Center for Environmental Excellence by AASHTO" website (15)

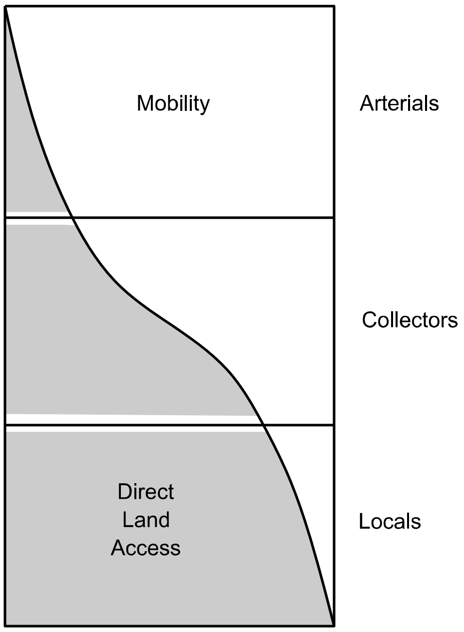
2.4 Functional Classification

2.4.1 Relationship to Roadway Design

Motor vehicle travel involves a series of distinct travel movements. The six recognizable stages in most trips include main movement, transition, distribution, collection, access, and termination. Each of the six stages of a typical trip is handled by a separate facility designed specifically for the function.

The functional classification concept is one of the most important determining factors in roadway design. In this concept, roads are grouped by the character of service they provide. Functional classification recognises that the public road network in Abu Dhabi serves two basic and often conflicting functions — access to property and travel mobility. See Figure 2-1. Each road or street will provide varying levels of access and mobility, depending upon its intended service. The overall objective of the functional classification system, when viewed in its entirety, is to yield an optimum balance between its access and mobility for all users (pedestrians, cyclists, motor vehicles) purposes. When achieved, the benefits to all users will be maximised.

Figure 2-1: Relationship of Functionally Classified Systems in Serving Traffic Mobility and Land Access



Source: (1)

The functional classification system provides the framework for determining the geometric design of individual roadways and streets. Once the function of the roadway facility is defined, the designer can select an appropriate design speed, roadway width, roadside safety elements, amenities, and other design values. The Abu Dhabi *Road Geometric Design Manual* is based upon this systematic concept to determine roadway design.

Table 2-2 summarises the principle features for each functional classification. For additional guidance on functional classifications, see AASHTO *A Policy on Geometric Design of Highways and Streets* (1).

Table 2-2: Characteristics of Roads by Class

Characteristic	Road Classifications				
	Freeways	**Expressways**	**Arterials**	**Collectors**	**Local Roads**
Land Service	No Access	Restricted access from service roads	Land access a secondary consideration	Land access and traffic movement of equal importance	Land access the primary consideration
Traffic Mobility	Optimum mobility	Optimum mobility	Traffic movement primary consideration	Land access and traffic movement of equal importance	Traffic movement secondary consideration
Pedestrians	None	Separated Pathway	Sidewalks	Sidewalks	Sidewalks
Public Transit Service	Express	Express	Express and Local	Local	Local
Cyclists	None	Separated Pathway	Cycle Track/Cycle Lane	Cycle Track/Cycle Lane	Cycle Track/Cycle Lane/Shared Roadway
Typical Length of Road	Unlimited	Unlimited	Urban: < 5 km Rural: as required	Urban: < 2 km Rural: as required	Urban: < 1 km Rural: as required
Typical Intersection/ Interchange Spacing	Urban: 1.5 km Rural: >3 to 5 km	Urban: > 1 km Rural: 3 to 5 km	Urban: >400 m Rural: >1.5 km	Urban: >200 m Rural: 400 m to 1.5 km	Urban: as required Rural: > 200 m
Nature of Traffic Flow	Free flow	Free flow	Uninterrupted flow except at intersections	Interrupted flow	Interrupted flow
User Type	Passenger Cars, Trucks and Buses	Passenger Cars, Trucks and Buses	Rural: Passenger Cars, Trucks* and Buses	Rural: Passenger Cars, Trucks* and Buses	Passenger Cars, and Transit Bus**
			Urban: Pedestrians, Transit Buses, Cyclists, Passenger Cars and Trucks	Urban: Pedestrians, Transit Buses, Cyclists, Passenger Cars and SU Trucks*	Urban: Pedestrians, Cyclists, Passenger Cars
Connect To	Freeways Expressways Arterials	Freeways Expressways Arterials	Freeways Expressways Arterials Collectors	Arterials Collectors Local roads	Collectors Local roads

Notes:

* *In industrial areas, collectors should accommodate all types of vehicle.*

** *In industrial and rural areas, trucks may require consideration.*

Source: (16) (8) *Revised*

2.4.2 Function of Roadway System for Rural Areas

The rural functional classification includes the following:

1. Principal Arterial System. The rural principal arterial system consists of a connected rural network of continuous routes having the following characteristics:

    - Serve corridor movements having trip length and travel density characteristics indicative of substantial inter- and intra-Emirate wide travel.
    - Serve all or virtually all urban areas with population of 30,000 and over, and a large majority of those with population of 20,000 and over.
    - Provide an integrated network without stub connections, except where unusual geographic or traffic flow conditions dictate otherwise (e.g. international boundary connections and connections to coastal cities).

    The rural principal arterial system is divided into the following subcategories:

    a. Freeways. The freeway is the highest level of an arterial. These facilities are characterised by full control of access, high design speeds, and a high level of driver comfort and safety. For these reasons, freeways are considered a special type of road within the functional classification system, and separate design criteria have been developed for rural freeways.

    b. Other Principal Arterials. These facilities include expressways and other multilane arterials, with or without a median. Partial control of access is desirable along these facilities. The access is limited by governing regulations.

2. Minor Arterials. Rural minor arterial system, in conjunction with the principal arterial system, should form a rural network having the following characteristics:

    a. Links. Arterials should link cities, larger towns, and other traffic generators that are capable of attracting travel over similarly long distances.

    b. Spacing. Arterials should be spaced consistent with population density so that all developed areas of Abu Dhabi are within a reasonable distance of an arterial road.

    c. Service. Arterials should provide service to roads with trip length and travel density greater than those predominantly served by rural collectors or local systems. Minor arterial routes have relatively high overall speeds and minimum interference to through traffic.

3. Collector Roads. The function of rural collector roads is to collect traffic flow from the rural local roads to the rural arterials and to distribute traffic flow from arterials back to the local roads. Access to properties is normally allowed on collector roads. In rural areas, the function of collector is twofold: To provide access to adjacent land uses and to carry traffic into areas with sparse development. Collector roads are further defined as follows:

a. Major Collector Roads. These routes serve larger towns and significant traffic generators (e.g. shipping ports, mining areas) that are not served by an arterial, expressway, or freeway. They also link these locations with higher classified routes.

b. Minor Collector Roads. These roads are spaced at intervals consistent with the traffic population density to accumulate traffic from local roads, provide service to smaller communities, and link locally important traffic generators.

4. Local Roads. Rural local roads generally have the following characteristics:

- constitute the rural roads not designated as part of a higher classification,
- serve primarily to provide access to adjacent lands and connections to higher classified routes, and
- provide service to motorists who travel relatively short distances as compared to collectors or other higher classified routes.

Table 2-3 provides typical percentages of rural functional classifications.

Table 2-3: Guidelines for Rural Functional Systems (U.S.)

System	Percentage of Total Rural Road Length
Principal Arterial System	2% – 4%
Principal Arterials and Minor Arterial System	6% – 12%
Collector Road System	20% – 25%
Local Road System	65% – 75%

Source: (1)

2.4.3 Function of Roadway System for Urban Areas

Urban areas are associated with cities and towns. AASHTO refers to suburban areas as the area surrounding urban centres, which are usually residential areas on the outskirts of a large city or town. Suburban areas may contain company facilities and industrial parks, but are primarily filled with housing and associated retail businesses. *Note: In Abu Dhabi, the term "suburban" is not used in its planning and design documents. Consequently, this Manual does not provide design criteria for suburban areas.*

The functional classification of urban roads and streets is usually less clear than that of rural roads, as urban roads and streets generally are flanked by dense development that requires frequent access at the boundary of the road. Historical requirements for curb side parking and other uses (e.g. public transport routes, pedestrians, and/or cycle routes) further complicate functional definitions.

Most urban arterial roads continue to function as major through traffic routes, but the management of these roads often requires space to be dedicated to pedestrians, public transport, and cycle use in preference to private car travel. There is also a trend on outer urban roads for speed limits to be lowered to address pedestrian safety issues while sections of inner city streets (formerly through arterial routes) are sometimes converted to pedestrian areas or shared zones. Consequently, the function of particular sections of road may change over time in accordance with community values. To better define urban roads and streets based on context sensitive street design, the Abu Dhabi Urban Planning Council defines urban streets with a two-name convention as described in Section 2.4.4. The first name is the context name, based on the land use context, and the second is the street family name, based on the transport capacity.

Urban functional classifications are defined as follows:

1. Principal Arterial System. In general, the urban principal arterial system carries the highest traffic volumes and accommodates the greatest trip lengths. The following are the main characteristics for streets and roads of the urban principal arterial system:

    - serves the major traffic movements within urbanised areas connecting central business districts, outlying residential areas, major intercity communities, and major urban centres;
    - serves a major portion of the trips entering and leaving the urban area, as well as the majority of the through traffic desiring to bypass the city; and
    - provides continuity for all rural arterials that intercept the urban area.

    The principal arterial system is segregated as follows:

    a. Freeways and Expressways. Freeways and expressways may be connecting links in the urban area, and they may be extensions of rural or other principle arterials. These routes may traverse the urban area from one boundary to another or may simply connect to another connecting link. In addition, freeways and expressways may provide access to circumferential routes around the city or provide links to the central city.

    b. Other Principal Arterials. These routes consist of a connected urban network of continuous routes having the following designations and characteristics:

        - provide service to, through, or around urban areas from rural arterial routes, and may be connecting links;
        - serve generally as an extension of a rural arterial road;
        - typically the access is controlled by regulation;
        - provide for an integrated network serving the entire urban area;
        - provide sidewalks for pedestrians and cycle tracks/cycle lanes for cyclists, if needed; and
        - carry the most important transit routes.

2. Minor Arterials. Minor arterials include all arterials not classified as principal arterials. These routes have the following general characteristics:

    - place more emphasis on land access than principal arterials;
    - provide service for trips of moderate length and at a somewhat lower level of mobility than urban principal arterial routes;
    - provide access to geographic areas smaller than those served by the higher system;

- provide intra-community continuity, but will not, for example, penetrate neighbourhoods;
- provide sidewalks for pedestrians and cycle tracks/cycle lanes for cyclists; and
- transit routes may be express or local.

3. Collector Streets. In urban areas, collector streets serve as intermediate links between the arterial system and points of origin and destination. These facilities typically have the following characteristics:

    - provide both access and traffic circulation within residential neighbourhoods and commercial/industrial areas,
    - may penetrate residential neighbourhoods or commercial/industrial areas to collect and distribute trips to and from the arterial system,
    - provides sidewalks for pedestrians and cycle tracks/cycle lanes for cyclists, and
    - transit routes are generally limited to local routes.

4. Local Streets. The streets functionally classified as urban local streets generally have the following characteristics:

    - constitutes the urban streets not designated as part of a higher classification;
    - serves primarily to provide direct access to abutting land and higher order systems;
    - offers the lowest level of mobility and highest land access services;
    - discourages through traffic movements;
    - encourages use by pedestrians;
    - provides cycle tracks, cycle lanes, and/or shared lanes for cyclists; and
    - transit routes are limited to local routes.

Table 2-4 provides typical percentages of urban functional classifications.

Table 2-4: Guidelines for Urban Functional Systems (U.S.)

System	Percentage of Total Urban Street Length
Principal Arterial System	5% – 10%
Principal Arterials and Minor Arterial System	15% – 25%
Collector Road System	5% – 10%
Local Road System	65% – 80%

Source: (1)

2.4.4 Urban Streets

Urban streets in the Emirate of Abu Dhabi serve many functions, and street classifications must reflect more than the simple balance between automobile movement and access. Many streets in Abu Dhabi must accommodate both a high degree of automobile movement and a high degree of accessibility. To better define the various modes of transportation within a municipality, the Abu Dhabi Urban Planning Council defines municipal streets in the *Abu Dhabi Urban Street Design Manual* (4), as follows:

1. Boulevard. A boulevard is a high vehicle capacity 3+3 street (three lanes in each direction). Boulevards may have frontage lanes. Existing streets with 3+3 lanes or more are also classified as boulevards.
2. Avenue. An avenue is a medium vehicle capacity 2+2 (two lanes in each direction) street. Avenues may have frontage lanes.
3. Street. A street is a low vehicle capacity 1+1 street (one lane in each direction). They typically have low traffic volumes and low speeds.
4. Access Lane. An access lane is a very low volume capacity 1+1 street. They typically have very low traffic volumes and very low speeds.

Table 2-5 illustrates the relationship between the Abu Dhabi street family and AASHTO functional classification systems.

Table 2-5: Relationship Between Street Family and Functional Classification Systems

Street Family	AASHTO Functional Classification			
	Principal Arterial	**Minor Arterial**	**Collector**	**Local**
Boulevard	X	X		
Avenue		X	X (major collectors)	
Street			X (minor collectors	X
Access Lane				X
Shared Street				X

Source: (4)

2.5 Speed

2.5.1 Design Speed

The project design speed is the selected speed for each project that will establish criteria for several design elements, including horizontal and vertical curvature, superelevation, sight distance, etc. The assumed design speed should be a logical one with respect to the topography, anticipated operating speed, the adjacent land use, and functional classification. Except for local streets where speed controls are frequently included intentionally and on non-freeway urban streets, the designer should make every effort to use as high a design speed as practical to attain a desired degree of safety, mobility, and efficiency. The selected design speed should reflect the actual operating speeds to ensure the safe and efficient movement of traffic. The selected design speed should not be less than the intended posted speed limit. Table 2-6 shows the recommended design speeds for each roadway functional classification for both rural and urban roads and posted speed limits for urban roads..

Table 2-6: Design Speed Guidelines

Road Class	Design and Posted Speeds (km/h)				
	Rural Roads Design Speed			Urban Streets	
	Maximum	Minimum	Mountainous	Posted Speed	Design Speed
Access Lane	n/a	n/a	n/a	20	30
Local Road/Street	80	60	40	30	40
Collector/ Avenue	100	80	50-60	40-50	60
Arterial/ Boulevard	110	90	60	80	80
Truck Routes	120	100	80	n/a	n/a
Expressway	120	100	80	100-120	100-120
Freeway	140	120	80	120-140	120-140

Note: As per the Executive Council resolution 1075/ECAS dated 19-07-2017, speed margin between the posted speed and the design is removed on the main roads where speed cameras to be located; therefore posted speeds on the main roads where speed cameras are installed shall be equal to the design speed.

2.5.2 Average Running Speed

The running speed is the speed of over a specified section of road. It is equal to the distance travelled divided by the running time (i.e. the time the vehicle is in motion). The average running speed is the distance summation for all vehicles divided by the running time of all vehicles. By measuring average running speeds at various locations along an existing section of road, the designer can identify those locations in similar terrain where speeds are not uniform with the rest of the section and make decisions for improvements.

The average running speed is used to determine the minimum acceptable acceleration or deceleration distances for freeway entrance and exit ramps. For all other geometric design elements, use the selected design speed to determine the appropriate design criteria.

2.5.3 Average Travel Speed

The average travel speed is speed over a specified section of road. It is equal to the length of the roadway segment divided by the average travel time of all vehicles traversing the segment, including time vehicles are not in motion. Average travel speed is equal to the space mean speed.

2.5.4 Operating Speed

The operating speed is the speed that drivers are observed operating their vehicles during free-flow conditions. In practice, the term “operating speed” is commonly used to characterise prevailing vehicular speeds on a road segment, either through field measurements of speed or through informal field observations. Although no precise percentile is used to define operating speed, typically it will be the 85th percentile of the actual travel speeds.

2.5.5 Posted Speed Limit

Posted speed limit is the legal speed limit for a road that is determined by statutes or based on an engineering and traffic investigation. Speed limits applied to roads have an effect on vehicular speeds and, in particular, tend to restrain the fastest drivers. These posted speeds are mandatory and are clearly displayed to road users. Nevertheless, it is important to provide a margin of safety for those vehicles whose drivers choose to travel faster than the speed limit. Table 2-7 indicates the posted speed that is appropriate for a given design speed. Posted speeds are typically 20 km/h below the design speed for high-design speeds (80 km/h or greater) and 10 km/h below the design speed for low-design speeds (60 km/h or less). If the 85th percentile speed is greater than the posted speed limit, this is considered an enforcement issue and is not a design issue.

Table 2-7: Posted Speeds

Design Speed (km/h)	Posted Speed (km/h)
30 – 40	30 – 40
50	40
60	50
70	60
80	80
90	90
100	100
120	120
140	140

Note: As per the Executive Council resolution 1075/ECAS dated 19-07-2017, speed margin between the posted speed and the design is removed on the main roads where speed cameras to be located; therefore posted speeds on the main roads where speed cameras are installed shall be equal to the design speed.

2.5.6 Truck/Car Speed Relationship

Rural posted speed for trucks and other heavy-duty vehicles are less than the posted speed for passenger cars and other light vehicles (typically 80 km/h or less). The *Highway Capacity Manual 2010* (17) defines heavy vehicles as any vehicle with more than four wheels on the ground during normal operations. These vehicles are generally categorised as trucks, buses, and recreation vehicles. They typically have less acceleration and require more room for manoeuvring, lane changing, and braking.

Trucks typically accelerate more slowly from a stop condition and experience greater difficulty in maintaining desirable speeds on long, steep upgrades than automobiles. Truck characteristics affecting speed on upgrades include mass, power, aerodynamic resistance, drive-train-to-gear ratios, and tires.

The number heavy vehicles are converted to equivalent passage cars for capacity analysis purposes. The equivalent ratio is determined based on speed, terrain, power, etc., of the heavy vehicle. See the *Highway Capacity Manual 2010* (17) for applicable equivalency ratios.

2.6 Design Vehicles

2.6.1 Vehicle Dimensions

The physical and operational characteristics of vehicles using the road are important controls in roadway design. Design criteria may vary according to the type of vehicle and the volume of each type of vehicle in the traffic stream. The design vehicle affects the radius returns, left-turn radii, lane widths, median openings, turning roadways, and sight distances at intersections.

For urban areas, Design vehicles represent the vehicles which must be regularly accommodated at junctions without encroachment into the opposing traffic lanes. Design vehicles applicable to the USDM street family descriptions comprise WB-12 for Boulevards and Avenues and Single Unit Bus/Truck (SUM) for streets and access lanes. City-Bus M is applicable for streets with designated public bus routes and/or school bus routes at school zones. Bus routes need to be discussed and agreed with DMT and their requirements incorporated into the design. Other design vehicles may also need to be considered, including speed control vehicles, control vehicles and non-motorised vehicles, depending on specific project requirements. Reference is to be made to the USDM for further details.

Urban streets shall be designed to accommodate the occasional use of emergency vehicles (such as a fire truck), although this is not considered the general design vehicle.

Vehicular characteristics that impact design include:

1. Size. Vehicular sizes determine lane and shoulder widths, vertical clearances and, indirectly, roadway capacity calculations.
2. Off tracking. The design of intersection turning radii, travelled way widening for horizontal curves, and pavement widths for interchange ramps are usually controlled by the largest design vehicle likely to use the facility with some frequency.
3. Storage Requirements. Turn-bay storage lengths, bus turnouts, and parking lot layouts are determined by the number and types of vehicles to be accommodated.
4. Sight Distance. Eye height and braking distances vary for passenger cars and trucks, which can impact sight distance considerations.
5. Acceleration and Deceleration. Acceleration and deceleration rates often govern the dimensioning of such design features as speed-change lanes at intersections or interchange ramps, climbing lanes, or passing lanes.
6. Vehicular Stability. Certain vehicles with high centres of gravity may be prone to skidding or overturning, affecting design speed selection, and superelevation design elements.

Table 2-8 and Table 2-9 present design vehicular dimensions and minimum turning radii for design vehicles used in Abu Dhabi. These tables can be used to input vehicle dimensions into turning template software (e.g. AutoTurn). Figure 2-2 and Figure 2-3 illustrate the application of the basic dimensions for combination trucks.

2.6.2 Turning Templates

The design vehicle affects the radius returns, left-turn radii, lane widths, median openings, turning roadways, and sight distances at an intersection. Figure 2-4 provides turning characteristics of typical design vehicle. The designer should use the appropriate turning software programs to check their designs. Include a copy of swept path design when submitting plans to the DMT for review. The basic design vehicles used by Abu Dhabi for design include:

- Passenger Car (P)
- Single Unit Truck (SU-9 and SU-12)
- City Transit Bus (CITY-BUS)
- Semitrailer Intermediate (WB-12 and WB-15)
- Interstate Semitrailer (WB-20)
- Double Semitrailer/Trailer (WB-33D)

Table 2-8: Minimum Turning Radii of Design Vehicles

Design Vehicle Type	Symbol	Minimum Design Turning Radius (m)	Centreline(1) Turning Radius (CTR) (m)	Minimum Inside Radius (m)
Passenger Car	P	7.26	6.40	4.39
Single Unit Truck	SU-9	12.73	11.58	8.64
Single Unit Truck	SU-12	15.60	14.46	11.09
City Transit Bus	CITY-BUS	12.50(2)	11.52	5.30**
Intermediate Semitrailer	WB-12	12.16	10.97	5.88
Intermediate Semitrailer(3)	WB-15	13.66	12.50	5.20
Interstate Semitrailer(3)	WB-20	13.66	12.50	0.59
Double Trailer Combination(3)	WB-33D	18.25	17.04	4.19

Source: (1) *Revised*

(1) *The turning radius assumed by a designer when investigating possible turning paths and is set at the centreline of the front axle of a vehicle. If the minimum turning path is assumed, the CTR approximately equals the minimum design turning radius minus one-half the front width of the vehicle.*

(2) *Based on dimensions provided by the DMT Public Transport Division.*

(3) *Only use this design vehicle in rural areas.*

For other design vehicles, see the AASHTO *A Policy on Geometric Design of Highways and Streets* (1). Note that the other design vehicles generally have better performance (e.g. off tracking) than the design vehicles listed above.

Table 2-9: Design Vehicle Dimensions

Dimensions (m)	**Typical kingpin to center of rear axle**		—	—			—		7.77	11.40	13.87	12.34
	Wheelbases	**WB_3**	—	—			—		—		—	12.19
		T	—	—			—		—		—	3.05[4]
		S	—	—			—		—		—	1.37[4]
		WB_2	—	—			—		8.38	10.80	13.84	12.19
		WB_1	3.10	6.10	7.62		7.00[5]		3.81	4.50	6.58	3.72
	Overhang	**Rear**	1.10	1.83	3.20		2.44		0.76[2]	0.60[2]	1.44[3]	1.37[3]
		Front	0.80	1.22	1.22		2.87[5]		0.91	0.91	1.22	0.71
	Overall	**Length**	5.00[1]	9.14	12.04		13.70[5]		13.87	16.80	22.40	34.75
		Width	2.13	2.44	2.44		2.50[5]		2.44	2.59	2.59	2.59
		Height	1.30	3.35-4.11	3.35-4.11		3.20		4.11	4.11	4.11	4.11
Symbol			P	SU-9	SU-12		CITY BUS		WB-12	WB-15	WB-20[2]	WB-33D[2]
Design Vehicle Type			Passenger car	Single-unit truck	Single-unit truck	**Buses:**	City Transit Bus	**Combination trucks:**	Intermediate semitrailer	Intermediate semitrailer[6]	Interstate semitrailer/trailer[6]	Double-semitrailer/trailer[6]

Source: (1) *Revised*

Notes:

WB_1, WB_2 and WB_3 are the effective vehicle wheelbases or distances between axle groups, starting at the front and working towards the back of each unit.
S is the distance from the rear effective axle to the hitch point or point of articulation.
T is the distance from the hitch point or articulation measured back to the center of the next axle or center of tandem axle assembly.

(1) Passenger cars may range from 5.0 m (Austroads Design Vehicles and Turning Path Templates) to 5.79 m (AASHTO).
(2) Design vehicle with 16.15-m trailer.
(3) Combined dimension is typically 3.81 m. This is overhang from the back axle of the tandem axle assembly.
(4) Combined dimension is typically 3.81 m.
(5) Public Transport Division bus dimensions.

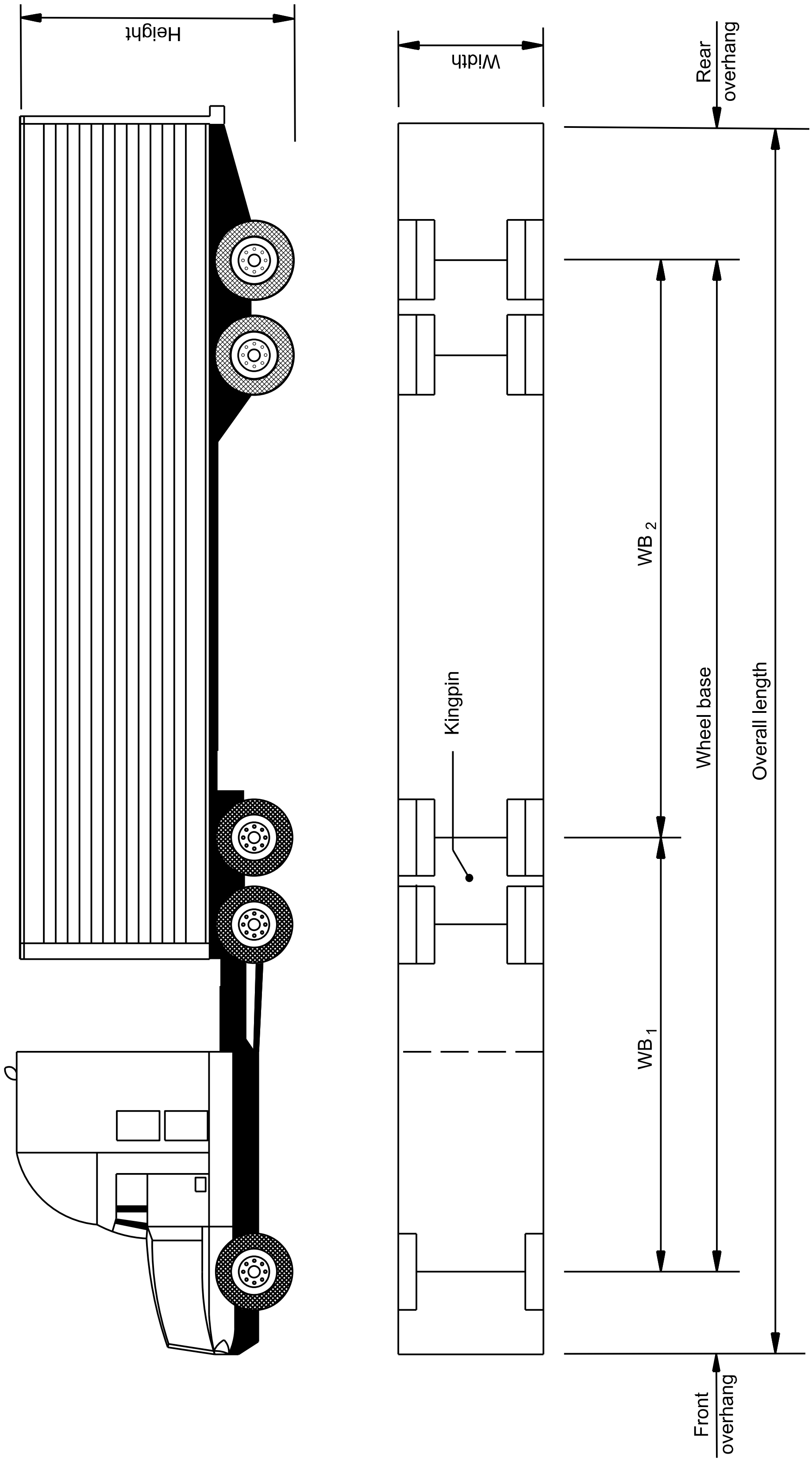
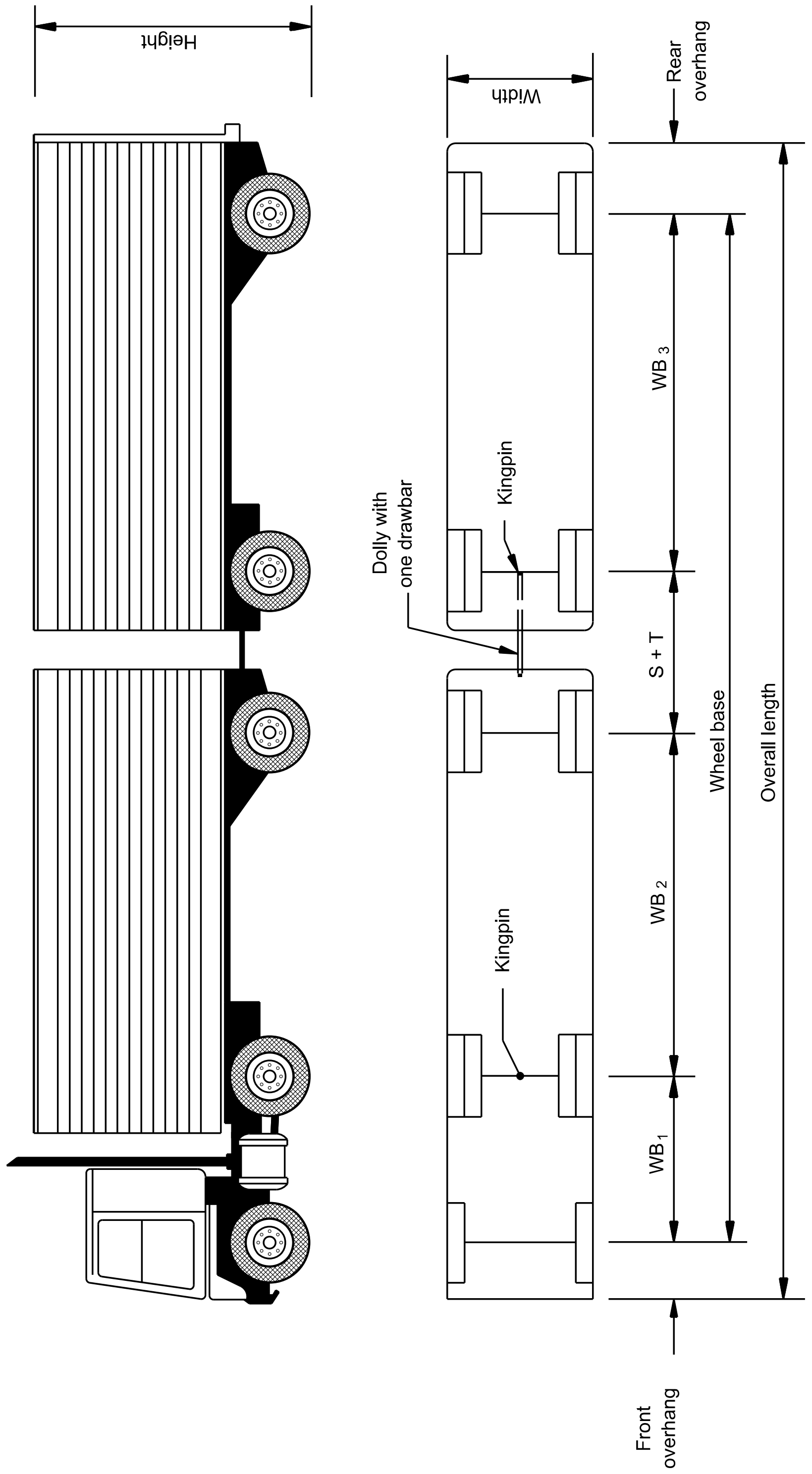


Figure 2-2: Basic Dimension of Tractor–Semitrailer Vehicle

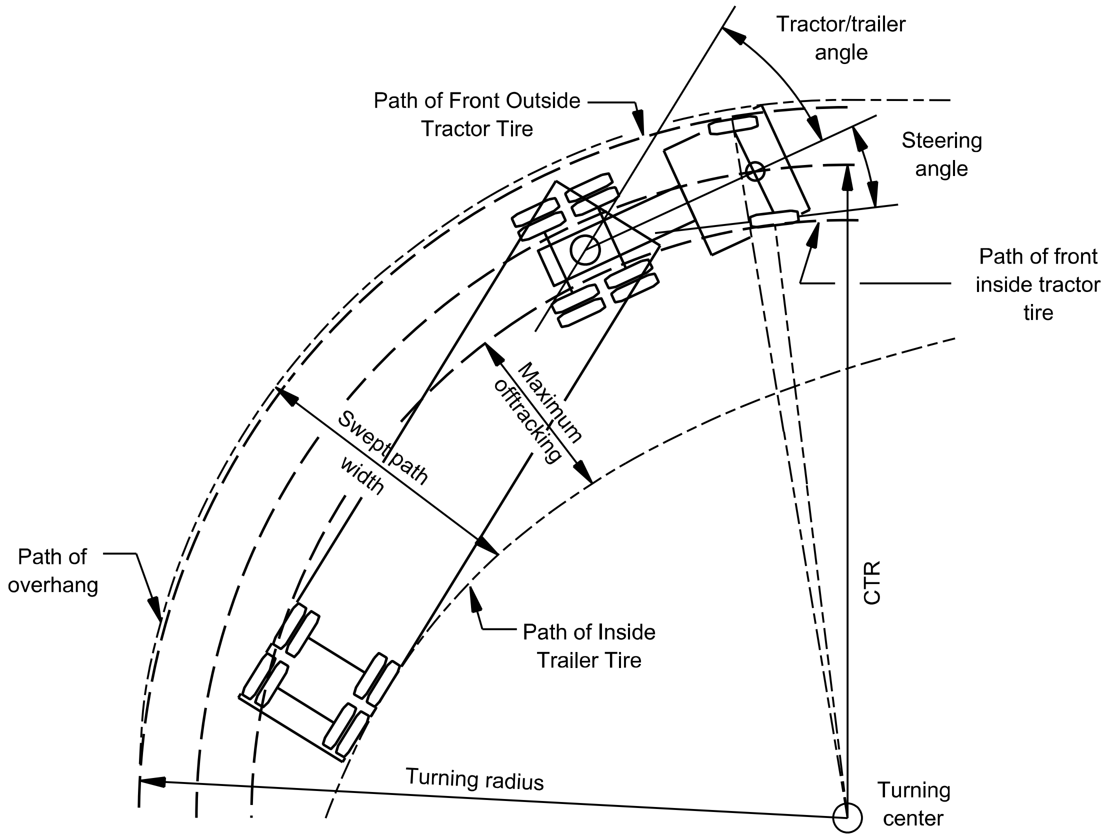
Figure 2-3: Basic Dimension of Tractor–Semitrailer Vehicle/ Trailer Vehicle (Double Trailer)



S = Distance from the rear effective axle to the hitch point or point of articulation

T = Distance from the hitch point or articulation measured back to the centre of the next axle or centre of tandem axle assembly.

Figure 2-4: Turning Characteristics of a Typical Semitrailer Design Vehicle



Definitions:

1. Turning Radius. The circular arc formed by the turning path radius of the front outside tire of a vehicle. This radius is also described by vehicular manufacturers as the “turning curb radius.”

2. CTR. The turning radius assumed by a designer when investigating possible turning paths. It is set at the centre of the front axle of a vehicle.

3. Off tracking. The difference in the paths of the front and rear wheels of a vehicle as it negotiates a turn. The path of each rearward tire of a turning vehicle does not coincide with that of the corresponding forward tire. This phenomenon is shown in the drawing above.

4. Swept Path Width. The amount of roadway width that a vehicle covers in negotiating a turn equal to the amount of off tracking, plus the width of the vehicle. The most significant dimension affecting the swept path width of a tractor/semitrailer is the distance from the kingpin to the rear trailer axle or axles. The greater this distance, the greater the swept path width.

5. Steering Angle. The maximum angle of turn built into the steering mechanism of the front wheels of a vehicle. This maximum angle controls the minimum turning radius of the vehicle.

6. Tractor/Trailer Angle. The angle between adjoining units of a tractor/semitrailer when the combination unit is placed into a turn. This angle is measured between the longitudinal axes of the tractor and trailer as the vehicle turns. The maximum tractor/trailer angle occurs when a vehicle makes a 180° turn at the minimum turning radius and is reached slightly beyond the point where a maximum swept path width is achieved.

Source: (1)

2.6.3 Acceleration and Deceleration Rates

Acceleration and deceleration rates of trucks are often critical parameters in determining the road design. These rates often govern the dimensions of such design features as intersections, freeway ramps, climbing or passing lanes, and turnout bays for buses. Trucks typically accelerate more slowly from a stop condition and experience greater difficulty in maintaining desirable speeds on long, steep upgrades than passenger cars. Based on their acceleration performance, the passenger cars seldom control design.

Trucks generally require longer deceleration distances than passenger cars. On vertical curves, the higher eye height for trucks often compensates for this longer distance. However, at locations with lateral sight distance restrictions (inside of horizontal curves), the benefits of the higher eye level may be lost. Therefore, truck deceleration considerations or other remedial measures (e.g. signing and higher friction surfaces) may be required.

The AASHTO *A Policy on Geometric Design of Highways and Streets* (1) provides passenger car deceleration and acceleration rates. Chapter 6 “Vertical Alignment” provides data on truck acceleration and deceleration rates.

2.7 Transportation Demand

To properly design the roadway facility, the road designer must determine the appropriate transportation/traffic related parameters to use for the design of the facility. Selection of the appropriate parameter is dependent upon several factors. The designer must ensure the parameter value is appropriate for their design. For additional guidance on selection of the applicable parameters, see the *Highway Capacity Manual 2010* (HCM) (17).

2.7.1 Traffic Volume Demand Parameters

Traffic volume demand parameters are necessary to produce traffic demand estimates for use in planning, project alternative evaluation, and selection, environmental studies, designs that lead to construction, transportation network improvement, and pavement design. In order to determine traffic volume demand parameters, it is important to identify the different road network facility types (i.e. definitions and attributes) that currently exist. The different types of road network facilities in Abu Dhabi are classified in Section 2.4. The designer also needs to consider all user types (e.g., pedestrians, transit users, cyclists, motor vehicles) in determining the demand.

2.7.2 Design Year Selection

The geometric design of a road should be designed to accommodate the expected user volumes predicted to occur during the life of the facility, assuming reasonable maintenance. This involves projecting the volumes to a selected future year. The forecasts for the farthest horizon year available in the STEAM model of DMT is to be utilized. In case it is not available for a particular area, the design year is 20 years from the expected construction completion date for new construction and reconstruction projects.

2.7.3 User Characteristics

Forecasting future demand volumes requires understanding of future land use, demographics, and changes in the transportation system. It is usually completed by using STEAM as described in the previous section. Current user data is used to predict the future ADT, peak hour(s), directional distribution, and traffic composition. These characteristics are discussed in the following sections.

Forecasts of future activity levels should include estimates of pedestrian and cyclist activity. Particular care must be used when forecasting pedestrian and cyclist volumes. Many times there is latent demand above observed pedestrian and cyclist volumes because pedestrian and cyclist facilities do not yet exist in the project area, are substandard, or do not provide complete connectivity to attractions. It is important to evaluate future land development, including any potential attractors (e.g. transit stops, schools, parks, retail uses) that may be located near moderate and high-density residential development.

2.7.3.1 Average Daily Traffic

The design of a road and its features should be based upon the traffic volumes and characteristics to be served. Information on traffic volume serves to establish the loads for the geometric road design. The data collected by agencies include traffic volumes for days of the year and time of day, as well as the distribution of vehicles by type and weight. This data also includes information on trends from which the designer may estimate the traffic to be expected in the future.

The basic measure of the traffic demand for a road is the average daily traffic (ADT) volume. The ADT is defined as the total volume during a given time period (in whole days), greater than one day and less than one year, divided by the number of days in that time period. The current ADT volume for a road can be readily determined when continuous traffic counts are available. When only periodic counts are taken, the ADT volume can be estimated by adjusting the periodic counts according to the season, month, or day of week.

2.7.3.2 Peak Hour Traffic

Capacity and other traffic analysis typically focus on the peak-hour volume as it represents the most critical period of operations and has the highest capacity requirements. The travel pattern on any road varies during hours of the day, day of the week, or time of year. Rural and recreational routes often show a wide variation in peak hour volumes. Urban streets, on the other hand, show less variation in peak-hour traffic.

A key design decision involves determining which hourly volume should be used for design. The selected design hourly volume (DHV) should not be so low that it is often exceeded, or so high that users would rarely be sufficient to make full use of the resulting facility. The selection of an appropriate hour for planning, design, and operational purposes is a compromise between providing an adequate level of service for all users for nearly every hour of the year and providing economic efficiency. For rural and recreational routes, the 30th highest hour is typically used for the DHV. For urban areas, the 100th hour is often used as the DHV. The designer should review the resulting conditions from the selected design hour to ensure the acceptability of the design (i.e. meets the needs of drivers, pedestrians, cyclists, and transit.).

2.7.3.3 Peak Hour Factor

Peak hour traffic volumes are critical in evaluating capacity and other parameters because they represent the most critical time period. It is the period during which demand is at its highest. The analysis of level of service is based on peak rates of flow occurring within the peak hour because substantial short-term fluctuations typically occur during an hour. Common practice is to use a peak

15-minute rate of flow. Flow rates are usually expressed in vehicles per hour, not vehicles per 15 minutes. The relationship between the peak 15-minute flow rate and the full hourly volume is given by the peak-hour factor (PHF) as shown in Equation 2.1:

$$\textbf{PHF = Hourly Volume/Peak Rate of Flow (within the hour)}$$

Equation 2.1: Peak Hour Factor

If 15-minute periods are used, the PHF is computed as:

$$PHF = V/(4 \times V_{15})$$

Equation 2.2: Peak Hour Factor (based on 15-minute period)

where: PHF = Peak Hour Factor
V = Peak Hour Volume (vph)
V_{15} = Volume during the peak 15 minutes of flow (veh/15 minutes)

The following example illustrates the application of the PHF:

Example 2.7–1

Given: One hour traffic count (7:00 – 8:00) along a rural freeway:

7:00 - 7:15: 170
7:15 - 7:30: 200
7:30 - 7:45: 220
7:45 - 8:00: 180

Problem: Compute the peak hour factor (PHF).

Solution: Determine the peak hour factor:

$PHF = 770/(4 \times 220) = 770/880 = 0.875$

Typical peak-hour factors for freeways range between 0.80 and 0.95. Lower factors are more typical for rural freeways or off-peak conditions. Higher factors are typical of urban peak-hour conditions.

2.7.3.4 Directional Distribution

For two-lane rural roads, the DHV is the total traffic in both directions of travel. In the design of roads with more than two lanes, on two-lane roads where important intersections are encountered, or where additional lanes are to be provided later, knowledge of the hourly traffic volume for each direction of travel is important.

A dual carriageway with a high percentage of traffic in one direction during the peak hours may need more lanes than a road having the same ADT, but an equal percentage of directional traffic. During peak hours on most rural roads, 55% to 70% of the traffic is travelling in the peak direction, with up to as much as 80% occasionally. Directional distributions of traffic vary enough between sites that two dual carriageways carrying equal traffic may have significantly different peak direction volumes.

Some studies have shown that urban radial routes may have up to 66% of the traffic travelling in the same direction during the peak hour. Unfortunately, this peak occurs in one direction in the morning and the opposite direction in the evening. Consequently, both directions of the facility must have capacity for the peak directional flow. Urban circumferential routes often have a 50/50 split during the peak hours for a more balanced flow.

The directional distribution of traffic during the DHV should be determined by making field measurements on the facility under consideration or on parallel and similar facilities.

In designing intersections and interchanges, the volumes of all movements occurring during the design hour is required. This information is necessary for both the morning and evening peak periods because the traffic pattern may change significantly from one peak hour to the other.

2.7.3.5 Traffic Composition

Vehicles of different sizes and weights have different operating characteristics and should be considered in road design. Trucks are generally slower, occupy more roadway space, and have a greater individual effect on traffic operations than do passenger vehicles. The effect on traffic operation of one truck is often equivalent to several passenger cars. The number of equivalent passenger cars equalling the effect of one truck is dependent on the roadway gradient and, for two-lane roads, on the available passing sight distance. Therefore, the larger the proportion of trucks in a traffic stream, the greater the equivalent traffic demand and the greater the road capacity needed.

For geometric design, it is essential to have traffic data on the types of vehicles expected for the roadway facility. These data generally indicate the major types of trucks and buses as percentages of all traffic expected to use the road. At intersections, the percentage of trucks during the morning and evening peak hours should be determined separately. Variations in truck traffic between the various traffic movements at intersections may be substantial and may influence the appropriate geometric layout.

2.7.3.6 Traffic Growth Rate

In studies that do not require running the regional travel demand forecasting model to estimate future traffic demands, the growth factor method can be used to estimate these demands. To establish future year forecasts, the growth factor method relies on the use of projected growth factors to existing year traffic data. The method assumes that the recent percentage of growth in traffic will continue in the future. These trend-line forecasts are most reliable for relatively short periods of time (5 years or less). Ideally, the growth factors are obtained based on traffic counts taken in the past few years. To estimate the growth factors, Equation 2.3 should be used.

$$\mathbf{EFD = BD*(1+i)^{n}}$$

Equation 2.3: Future Demand

where: FD = Future demand
BD = Base year demand
I = growth rate
n = number of years between base year and future year

2.7.4 Capacity and Operational Analyses

2.7.4.1 Highway Capacity Manual

Capacity is the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to transverse a point, or a uniform section of roadway during a given time period under prevailing road, environmental, traffic, and control conditions. The goal of the designer is to ensure the expected capacity of road reasonably meets the demand for all users of the facility (pedestrian, transit users, cyclists, motor vehicular users, trucks). There are several documents available to assist the designer in determining the capacity of a facility (e.g. United Kingdom's *Design Manual for Roads and Bridges*). In Abu Dhabi, the designer should follow the guidance provided in the *Highway Capacity Manual 2010* (HCM) (17).

The HCM was developed considering the following analysis areas:

- levels of analysis (operations, design, preliminary engineering, and planning);
- travel modes (automobile and other motorised vehicles, pedestrians, cyclists, and transit where it is part of a multimodal urban street facility);
- spatial coverage (points, segments, and facilities); and
- temporal coverage (undersaturated and oversaturated conditions).

The HCM provides tools to assist in estimating performance measures for individual elements of a multimodal transportation system, as well as guidance on combining those elements to evaluate larger portions of the system. Table 2-10 provides the system elements and the service measurements used in the HCM.

To more accurately determine roadway capacities, capacity methodologies have grown in complexity and cannot be reasonably documented (e.g. some methodologies require several iterations to reach a solution). Consequently, commercial software (e.g. Synchro, PASSER II, PASSER IV, CORSIM) is available to assist the road designer in running the complex methodologies presented in the HCM.

The HCM was developed for the conditions in North America and, in particular, the United States. Consequently, the designer should carefully evaluate the default values provided in the HCM to ensure they are accurate for the conditions in Abu Dhabi. The Abu Dhabi *Transportation Impact Study Guidelines* (18) provides the performance measures and criteria to be used in Abu Dhabi. The applicable performance measures are also provided in Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads,” Chapter 9 “Urban Streets,” and Chapter 10 “Intersections.”

Table 2-10: HCM Service Measures

System Element	HCM Chapter	Service Measure(s)				Systems Analysis Measure
		Automobile	Pedestrian	Bicycle	Transit	
Freeway Facility	10	Density	—	—	—	Speed
Basic freeway segment	11	Density	—	—	—	Speed
Freeway weaving segment	12	Density	—	—	—	Speed
Freeway Merge and Diverge Segments	13	Density	—	—	—	Speed
Multilane Road	14	Density	—	LOS Score	—	Speed
Two-Lane Road	15	Percent time spent following, speed	—	LOS Score	—	Speed
Urban Street Facility	16	Speed	LOS Score	LOS Score	LOS Score	Speed
Urban Street Segment	17	Speed	LOS Score	LOS Score	LOS Score	Speed
Signalised Intersection	18	Delay	LOS Score	LOS Score	—	Delay
Two-Way Stop	19	Delay	Delay	—	—	Delay
All-Way Stop	20	Delay	—	—	—	Delay
Roundabout	21	Delay	—	—	—	Delay
Interchange Ramp Terminal	22	Delay	—	—	—	Delay
Off-Street Pedestrian-Bicycle Facility	23	—	Space events	LOS Score	—	Speed

Source: (17)

2.7.4.2 Performance Measures

The performance measures and standards intended to be used for the purpose of evaluating and assessing the perception of roadway users' quality/level of service and capacity at planning, preliminary/detailed engineering design, and operation levels. They also provide an integrated multimodal approach to the analysis and evaluation of roadway transportation facilities from the points of view of automobile drivers, transit passengers, bicyclists, and pedestrians. Further, they provide thresholds for estimating and predicting acceptable performance levels in Abu Dhabi.

1. <u>Quality of Service</u>. Quality of service is a user based perception of how well a transportation service or facility operates. In other words, how the existing and potential users perceive the overall quality of service provided to them. It is a qualitative measure describing operational conditions within a traffic stream, based on service measures (e.g. speed and travel time, freedom to manoeuvre, traffic interruptions, comfort, safety, and convenience).

2. <u>Level of Service</u>. Level of service (LOS) is a quantitative stratification of quality of service. LOS reflects the quality of service as measured by a scale of user satisfaction and is applicable to each of the following modes that use roadways: automobiles and trucks, cyclists, pedestrians, and buses. LOS is given a letter of designation from A to F, with LOS A, representing the best operating condition while LOS F, represents an over-saturated and

heavily congested condition. Table 2-11 shows the typical operating conditions represented by these LOS. Each facility type generally has its own specific definitions for LOS. Procedures for computing operational and roadway factors to define for conditions that are other than ideal are applied through the level of service (LOS) concept.

In general, the designer should strive to provide the highest LOS practical. However, in urban environments, the designer needs to consider other factors (e.g. pedestrians, cyclists, transit users, level of development). In these cases, it may be more desirable to accept a lower LOS than otherwise may have been provided.

Table 2-11: General LOS Definitions of Automobiles

Level of Service	General Operating Conditions
A	Free flow
B	Reasonably free flow
C	Stable flow
D	Approaching unstable flow
E	Unstable flow
F	Breakdown flow

Note: Specific definitions of LOS A – F vary by facility type and are presented in the HCM.

Source: (17)

2.7.4.3 Principles and Categories of Planning and Operational Measures of Performance

- The performance measures should be measurable and expressed quantitatively over time to determine if the performance toward a goal is getting better or worse.
- The target for each performance indicator should be attainable and the desired result must be acceptable.
- The performance measures should be relevant and specific to each transportation facility.

The following categories of performance measures are considered inclusive and appropriate for Abu Dhabi transportation facilities. The proposed LOS criteria and thresholds for each roadway facility correspond to the relevant performance categories.

- volume-based measures
- delay-based measures
- density-based measures
- index-based measures
- speed and frequency-based measures

2.7.4.4 Principles for Acceptable Level of Performance

While determining the acceptable LOS threshold for Abu Dhabi, special and distinguished focus should be paid to how LOS thresholds relate to the following:

- greenhouse gas emissions and air quality
- energy consumption and vehicle operating costs
- network performance variations
- safety
- quality of life
- traffic congestion
- travel time reliability and costs
- vehicle operating costs

On the basis of the findings of the above relationships, the performance measures and LOS criteria for various transportation facilities are provided elsewhere in the applicable chapters of this *Manual*. However, it should be noted that the methodologies and procedures employed in determining the LOS standards for different transportation facilities are specified and described within the HCM.

2.7.4.5 Posted Speed

One of the important parameters used in capacity analysis is speed. Posted speeds are to be used in traffic capacity analysis (modelling).

2.7.4.6 Uninterrupted Roadway Capacity

Vehicle capacity of uninterrupted roadway facility is the maximum number of vehicles that can pass a given point during a specified period under prevailing roadway and traffic conditions. It is normally expressed in passenger cars/hour/lane. In the capacity and level of service analysis, heavy vehicles have to be converted to their passenger car equivalents. In general, the capacity analysis procedures that are used identify capacity values for a set of standard ideal or base conditions. A link capacity analysis is required for all links within the study area. Lane capacities for each link type in Abu Dhabi are provided in Table 2-12.

Table 2-12: Typical Lane Capacities in Abu Dhabi

Link Type	Lane Capacity (pc/hr/ln)
Freeway	2200
Expressway	2000
Arterial/Boulevard/Avenue (signalised, non-CBD)	1000
Arterial/Boulevard (signalised, CBD)	700
Collector/Avenue/Street	700
Ramp	1500
Rural Single Carriageway (1+1)	1100
Local Road/Street	600

Source: (18)

The results of the capacity analysis for links on arterial or collector roads with signal, stop, give-way, or roundabout control need to be discussed comparing to the relevant intersection analysis results. Free-flow links (e.g. freeways) may also be affected by weaving and merging. Critical results therefore need to be discussed and agreed upon with the Client.

For local (or residential) roads, a maximum traffic flow of 600 vehicles per direction per lane per peak hour applies. Traffic flows exceeding this value suggest other through-traffic using this road. A different function, for example as collector road demanding different designs and dimensions may be more appropriate. For a local road, other criteria (e.g. safety improvements, reduction of noise and pollution) are of higher importance and should guide design and dimensioning of the road, rather than capacity.

2.7.4.7 Saturation Flow Rate at Signalised Intersections

For signalised intersections, capacity is calculated as the product of saturation flow rate and effective green divided by the cycle length. Saturation flow rate is the maximum flow rate of vehicles that can be sustained across a signal stop line, assuming 100% green time and is expressed in passenger cars per hour green per lane. The value of saturation flow rate is required to estimate signalised intersection capacities and other associated performance measures using analytical procedures and microscopic simulation models.

To determine the lost-time at signalised intersections, the capacity of a signalised intersection movement is calculated as the product of saturation flow rate and the effective green divided by cycle length. The effective green for a given movement is calculated as the sum of the green and yellow times for the movement minus the total lost time. The lost time consists of two components:

- start-up lost time due to drivers' perception-reaction time at the beginning of green and,
- clearance (yellow) interval lost time at the end of yellow.

The default value for the start-up lost time is 2.0 seconds. The default value of the number of seconds used by vehicles during the yellow (sec.) is 3.0 seconds in the Emirate, even at intersections that have flashing green. The applicable All Red time is 2 seconds.

2.7.4.8 Pedestrian and Cyclist Speeds

Pedestrian and Cyclist's speeds are the most important capacity characteristics that need to be considered when designing pedestrian and cycle facilities. Walking speed is influenced by pedestrian density, gender, size of platoon, percentage of elderly population, disabled pedestrian population, and child pedestrian population. A walking speed of 1.2 m/s can be used as the design speed. However, use 0.9 m/s to 1.05 m/s at locations with high proportions of elderly and slower pedestrians.

The typical speed of cyclists is about 24 km/h for segregated cycle tracks and 16 km/h for shared facilities. Among the factors that affect cycle speed are the type of cycle, the cycle track, pavement surface type, weather conditions, the grade of the track, and the mix of other non-motorized users on the cycle track.

2.8 Environmental Considerations

Roads have wide-ranging effects in addition to providing traffic service to users. It is important that roads be considered as an element of the total environment. The term "environment" as used here, refers to the totality of humankind's surroundings (social, physical, natural, and synthetic). It includes the human, animal, and plant communities, and the forces that act on all three. The road can and should be located and designed to complement its environment and serve as a catalyst to environmental improvement.

The area surrounding a proposed road is an interrelated system of natural (e.g. wind, sand), synthetic, and sociologic variables. Changes in one variable within this system cannot be made without some effect on other variables. The consequences of some of these effects may be negligible, but others may have a strong and lasting impact on the environment, including the sustenance and quality of human life. Because road location and design decisions have an effect on the development of adjacent areas, it is important that environmental variables be given full consideration, as listed below:

- To present managers and decision makers with a clear assessment of potential impacts that a project may have on the overall environment;
- To apply to a project a methodology that assesses and predicts impacts and provides the (a) means for impact prevention and mitigation, (b) enhancement of project benefits, and (c) minimization of long-term impacts; and
- To provide a specific forum in which consultation is systematically undertaken in a manner that allows stakeholders to have a direct input to the environmental management process.

Also, exercise care to ensure that all applicable environmental requirements are met. For example, see Chapter 3 “Cross Section Elements” for guidance on blowing sand.

For additional guidance on environmental considerations, review the Abu Dhabi Planning Council ESTIDAMA website (12), Abu Dhabi *Environmental, Health, and Safety Manual for Main Roads* (19), and Abu Dhabi *Environmental Assessment for Road Projects Manual* (20).

2.9 Right-of-Way Considerations

The provision of right-of-way widths that accommodate construction, adequate drainage, and proper maintenance of a road is an important part of the overall design. Wide rights-of-way permit the construction of gentle slopes, resulting in greater safety for the motorist and providing for easier and more economical maintenance. The procurement of sufficient right-of-way at the time of the initial improvement permits the widening of the roadway and the widening and strengthening of the pavement at a reasonable cost as traffic volumes increase.

The minimum right-of-way width for all functional classifications will be the sum of the travel lanes, outside shoulders, and median width, if applicable, plus the necessary width for fill and cut slopes, or for roadside clear zones, whichever is greater. Desirably, the right-of-way width will accommodate the anticipated ultimate development of the roadway and utility services corridors. The geometric design tables in Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads,” and Chapter 9 “Urban Streets” provide the minimum right-of-way widths for roadways in Abu Dhabi. Determination of the final right-of-way width must be coordinated with the Client.

The right-of-way width should be uniform, where practical. In urban areas, variable widths may be necessary due to existing development; varying side slopes and embankment heights may make it desirable to vary the right-of-way width; and right-of-way limits will likely have to be adjusted at intersections and freeway interchanges.

In developed areas, it may be desirable to limit the right-of-way widths. However, the right-of-way width should not be less than that required for all the elements of the design cross sections, utility accommodation, and appropriate border areas. The client will review all official written requests, with justification, for departures from the minimum right-of-way requirements. For additional guidance on determining right-of-way widths in urban environments, see the *Abu Dhabi Urban Street Design Manual* (4).

Generally, embankment height between 1.5m-2.0m is used and can be more than 3.0m at drainage facilities and utility locations or Sabkha areas. Embankment heights shall be studied and minimized as much as possible unless any special cases such as Sabkha areas. Utility providers' requirements for existing/proposed networks shall be optimized. Drainage facilities shall be provided in natural stream/depressed areas in order to minimize their negative impacts on the road elevation, where applicable."

2.10 Access Control and Access Management

Access management is the careful consideration of the location, type, and design of vehicular access points to serve land development in a manner that helps minimise negative impacts while still providing sufficient accessibility to the land. It is a systematic and strategic response to the recognition that vehicular manoeuvres and volumes at each access point will have specific and accumulative consequences to the flow and safety of the road for all user types. It also involves roadway design applications (e.g. median treatments, auxiliary lanes), the design of intersections, and the appropriate spacing of traffic signals.

In its simplest form, access management is conflict management. In the flow of traffic from origin to destination, the fewer conflicts that occur during the driving experience, the less the delay and the lower the probability of a crash. When the speed of the vehicle or the frequency of access conflicts increases, the rate of conflict and the average frequency of crashes increase. Access management techniques work to reduce both the frequency and severity of traffic conflicts.

Access management is applied in a hierarchical manner. Freeways have the highest level of control where access is only allowed using well-spaced, well-designed directional ramps. The entire freeway has a non-traversable median to prohibit cross movements. As the functional hierarchy steps down, to expressway then arterial, more access is allowed, but must be well spaced and include design elements (e.g. turn lanes) and well-designed transitions. See Figure 2-1.

Access management in Abu Dhabi is a governmental program that sets standards for the location, type, and design of all access points to any public road in the entire Emirate. The program has been created and is implemented to help preserve the functional integrity of public roads to ensure that the mobility, efficiency, and safety of public roads are maintained for today and future generations. The Abu Dhabi *Access Management Policy and Procedures Manual* (21) defines 13 access categories based on the facility's functional classification and whether the facility is in an urban or rural environment. See the Abu Dhabi *Access Management Policy and Procedures Manual* for definitions and design requirements for each access category.

2.11 Non-Road Design Controls

The characteristics of drivers and vehicles significantly influence the selected design criteria. Where the driver and vehicle are properly accommodated, the safety and serviceability of the road system are enhanced. When they are not accommodated, inefficient operation may result.

2.11.1 Drivers

There are two types of drivers — typical and elderly drivers. The following provides guidance on these drivers:

1. Typical Driver. The appropriate considerations for drivers are already built into the applicable geometric design values (e.g. stopping sight distance, horizontal curvature, superelevation, roadway widths). Typical drivers vary widely in their operating skills, experience, intelligence, and physical condition. The road should be as forgiving as practical to minimise the adverse effects of driver errors. The following discusses certain principles and driver traits that should be incorporated into the roadway design:

    a. Information Processing. Drivers are limited in how quickly they can gather information, make a decision, and take action. They must process information related to lane placement, speed, traffic control devices, road alignment, roadside conflicts, and weather. If the amount, complexity, or clarity of the information is inappropriate or excessive, this may lead to driver error.

    b. Primacy. Certain driving functions are more important than others. In order of importance they are:

        i. Control. Activities related to the physical control of the vehicle via the steering wheel, brake, or accelerator.

        ii. Guidance. Activities related to selecting a safe speed and vehicular path on the road.

        iii. Navigation. Activities related to planning and executing a trip from point of origin to destination.

    The designer must be aware of the relative importance of these activities and ensure that the more important road information is properly conveyed to the driver. This could result in the decision to remove or relocate lower priority information, if it is likely to interfere with the higher priority information.

    c. Expectancy. Drivers are conditioned through experience and training to expect and anticipate what lies ahead on the road. If this driver expectancy is violated, it will increase the time needed by the driver to assess the situation and make the correct decision. These violations should be avoided. Where they are unavoidable, the designer should allow for increased warning time.

    d. Speed. Speed must be considered when accommodating the driver. Higher speeds reduce the visual field and restrict peripheral vision.

2. Elderly Driver. In general, the median age of drivers is increasing. This reality greatly emphasises the criticality of the relationship between the driver and the roadway environment. Although the opinions are not unanimous, there is general agreement that advancing age has an effect on an individual's perceptual, mental, and motor skills. It is important for the designer to be aware of the needs of the elderly driver and, where desirable, factor these needs into the roadway design.

2.11.2 Pedestrians and Cyclists

Roads throughout the Emirate are to be designed to emphasise family, hospitality, inclusiveness, and pedestrian access to neighbourhood facilities, including mosques and schools. All streets should be designed to accommodate pedestrians, transit users, cyclists, and motorists so that all modes offer an attractive choice.

Do not include provisions for pedestrians and cyclists along freeways. However, consider providing provisions for pedestrians and cyclists to safely cross the freeway at appropriate locations (e.g. pedestrian bridges).

Abu Dhabi Walking and Cycling Master Plan, Chapter 13 “Pedestrian and Cyclist Facilities”, and the *Abu Dhabi Urban Street Design Manual* (4) provide guidance on the design criteria for pedestrians and cyclists accommodations.

2.12 Road Safety

Highways should be designed to minimize driver decisions and to reduce unexpected situations. The number of crashes increases with an increase in the number of decisions required of the driver. Uniformity in highway design features and traffic control devices plays an important role in reducing the number of required decisions, and by this means, the driver becomes aware of what to expect on a certain type of highway.

The most significant design factor contributing to safety is the provision of full access control. Full access control reduces the number, frequency, and variety of events to which drivers must respond. The beneficial effect of this element has been documented in several reports. One of the principal findings of these studies is that highways without access control invariably had higher crash rates than those with access control.

The principle of full access control is invaluable as a means for preserving the capacity of arterial highways and of minimizing crash potential; however, this principle does not have universal application. Highways without control of access are essential as land service facilities, and the design features and operating characteristics of these highways need to be carefully planned so that they will reduce conflicts and minimize the interference between vehicles and still meet the needs of highway users.

Speed is often a contributing factor in crashes, but its role must be related to actual conditions at a crash site to be understood. It is improper to conclude that any given speed is safer than another for all combinations of the many kinds of drivers, vehicles, highways, and local conditions. For a highway with particularly adverse roadway conditions, a relatively low speed may result in fewer crashes than a high speed, but this does not necessarily mean that all potential crashes can be eliminated by low speeds. Likewise, vehicles traveling on good roads at relatively high speed may have lower crash involvement rates than vehicles traveling at lower speeds, but it does not necessarily follow that yet a higher speed would be even safer

When designing a highway, consideration should be given to the type and characteristics of the drivers expected to use the highway. Trip purposes (such as recreation, commuting to work, and through travel) are factors affecting the design to some extent. Trip purposes are related to the mix of vehicle types likely to use the highway, ranging from all passenger vehicles to a high percentage of heavy commercial vehicles. Where trips of one type predominate, the facility should be designed to fit the specific needs of that type of trip.

A highway with a median width of 15 m [50 ft] or more has a very low incidence of head-on collisions caused by vehicles crossing the median. With narrower medians, median barriers will eliminate head-on collisions, but at the cost of some increase in same-direction crashes because recovery space is decreased. Properly designed median barriers minimize vehicle damage and lessen the crash severity. However, if a narrow median with a median barrier is proposed on a high-speed highway, the design should include adequate shoulder widths in the median for emergency stops and emergency vehicle use.

Another study relating crashes to shoulder width, alignment, and grade found that crash rates on sections with curves or grades were much higher than on level tangent highway sections. Crashes are likely to occur where drivers are called upon to make decisions under circumstances where their vehicles are unable to respond properly, for example, where a truck is descending a grade. It would be logical to expect more crashes on grades and curves than on level tangent highways where driver decisions are needed less frequently and vehicles are fully responsive. However, design with tangent alignment can be overdone.

On extremely long tangents, drivers have a tendency to completely relax, especially after driving on a congested highway before entering a freeway. On some freeways, there has been concern over the number of crashes that occur when the driver apparently goes to sleep. It is considered highly desirable to provide gentle curvature and to avoid a fixed cross section for long tangent sections of roadway. This can be achieved by varying the median width, using independent roadway alignments, and taking advantage of the terrain, wherever practical. In addition, rumble strips can be added to shoulders to reduce run-off-the-road crashes caused by drivers falling asleep at the wheel. As the design of alignment, grade, and traveled-way cross section has improved, roadside design has also become increasingly important. Crashes involving single vehicles running off the road constitute more than one-half of all fatal crashes on freeways.

Basic to the concept of the forgiving roadside is the provision of a clear recovery area. Studies have indicated that on high-speed highways, a relatively level traversable width of approximately 9 m from the edge of the traveled way permits about 80 percent of the vehicles leaving the highway to safely stop or return to the roadway. Even though the 9-m width is not a “magic number” and the application of engineering judgment is necessary, the 9-m width has been used extensively as a guide for recovery zones.

In roadside design, two major elements should be controlled by the designer: roadside slopes and unyielding obstacles. On existing highways, the following priorities for treatment of roadside obstacles are recommended:

- Remove the obstacle or redesign it so it can be safely traversed.
- Relocate the obstacle to a point where it is less likely to be struck.
- Reduce severity of impacts with the obstacle by using an appropriate breakaway device.
- Redirect a vehicle by shielding the obstacle with a longitudinal traffic barrier and/or crash cushion.
- Delineate the obstacle if the above alternatives are not appropriate.

The design of guardrails and barrier systems has become a subject of considerable research. DMT’s Roadside Design Guide (TR-518) is one of many published reports that deal with this subject. These publications note that the treatment of end sections on guardrail or a barrier is of particular concern. The design team should assess their design against TR-518, the Emirate of Abu Dhabi Roadside Design Guide to ensure roadside safety is properly considered.

Communication with the motorist is probably one of the most complex problems for the designer. One of the best available tools concerning motorist communication is DMT's MUTCD which presents criteria for uniform application of signing, painted channelization, and pavement markings for all roads. A primary message of the MUTCD is the importance of uniformity. Highway users are dependent on traffic control devices (signs, markings, and signals) for information, warning, and guidance. So great is the dependence of highway users on such information that uniform, high-quality traffic control devices are necessary for safe, efficient use and public acceptance of any highway regardless of its excellence in width, alignment, and structural design.

All traffic control devices should have the following characteristics: (1) fulfill an important need, (2) command attention, (3) convey a clear, simple meaning, (4) command respect of road users, and (5) provide adequate response time. Four basic attributes of traffic control devices are essential to ensure that these devices are effective: design, placement, maintenance, and uniformity. Consideration should be given to these attributes during the design of a highway to ensure that the required number of devices can be kept to a minimum and that those that are needed can be properly placed.

A large proportion of crashes on rural highways occur at intersections. Several studies have been made at intersections with varying conditions, and the results vary according to conditions studied. Factors to be considered in designing an intersection are total traffic volume, amount of cross traffic, turning movements, type of highway, type of traffic control needed, design of the crossroad sight distance, and the utilization of islands and channelization. Various studies indicate improvements in safety at intersections can be accomplished by channelizing intersections, providing appropriate sight distances (including stopping, decision, and intersection sight distance), and providing safety refuge islands and sidewalks for pedestrians, lighting, signing, and traffic control devices. These concepts have been incorporated in this manual.Road Safety Audit

2.13 Road Safety Audit

Designers to ensure that all schemes to undergo relevant stage audits as per requirements set out in the Road Safety Audit Manual TR-540; designers to ensure consideration of safety requirements for ALL Road users.

The following Pre-Construction Audits to be undertaken:

Conceptual Design Stage Audits (Stage 0)

Preliminary Design Stage Audits (Stage 1)

Detailed Design Stage Audit (Stage 2)

The above stage audits must be closed by the relevant Abu Dhabi Transport Infrastructure authority to move to the next stage of Design or before "Issued for Conctruction" (IFC) Drawings are issued.

All pedestrian and cycling facilities improvements are subject to RSA in accordance with TR-540. RSA to be conducted by an independent 3rd party from the design team.

Refer to TR-540 Road Safety Audit Manual for further details and updates regarding the Road Safety Audit procedures and requirements.

3 CROSS SECTION ELEMENTS

3.1 Overview

This chapter provides general guidance on the design of cross section elements, including lane and shoulder widths, cross slopes, rumble strips, auxiliary lanes, curbs, medians, verges, side slopes, sand control, right-of-way, utilities, and tunnels. Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads,” and Chapter 9 “Urban Streets” provide the detailed design criteria for individual cross section elements.

Figure 3-1 through Figure 3-5 illustrate nomenclature and schematics of the basic elements of the roadway section for rural dual carriageways (freeways, expressways, arterials), urban dual carriageway (freeways), two-lane rural single carriageways (collectors, local roads), city boulevards, and city avenues, respectively. For other urban cross sections, see the *Abu Dhabi Urban Street Design Manual* (4).

3.2 Roadway Section

3.2.1 Carriageway

A single carriageway involves a single road that carries either a single direction of traffic or two directions of traffic, with each direction separated by specific longitudinal markings in the middle section of the roadway. A single carriageway may have one or more lanes of traffic in each direction, and may contain specific turn lanes at intersections.

A dual carriageway involves two sets of paved roadways, each serving traffic travelling in the opposite direction of the other, separated by a physical barrier (e.g. median or concrete barrier).

3.2.2 Travelled Way

The travelled way is the portion of the roadway for the through movement of vehicles, exclusive of shoulders. The travelled way consists of one or more travel lanes. For urban areas in Abu Dhabi, the term travelled way also includes the median, if applicable, between the through travel lanes.

3.2.2.1 Lane Width

Minimum travel lane widths vary depending on type of roadway and location of the facility. The geometric design tables in Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads” and Chapter 9 “Urban Streets” provide the minimum lane width based on the facility’s classification. The following provides the general lane width criteria:

- Travel lane widths for rural and urban freeways, rural arterials, and rural collectors are 3.65 m.

- For rural truck roads, lane widths of 3.75 m are preferred; however, lane widths of 4.0 m or 3.65 m may also be used. Do not use lanes widths wider than 4.0 m as drivers may try to use the wider lane as two travel lanes.

Figure 3-1: Rural Dual Carriageway Nomenclature

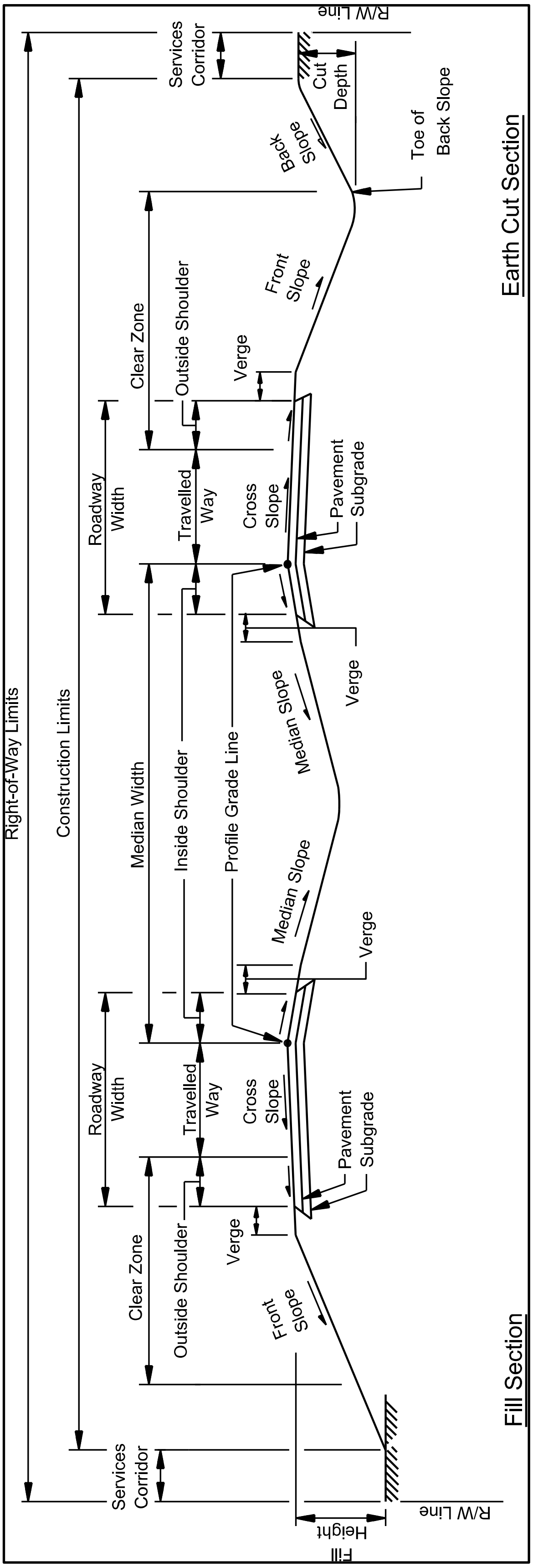


Figure 3-2: Urban Dual Carriageway (Freeways) Nomenclature

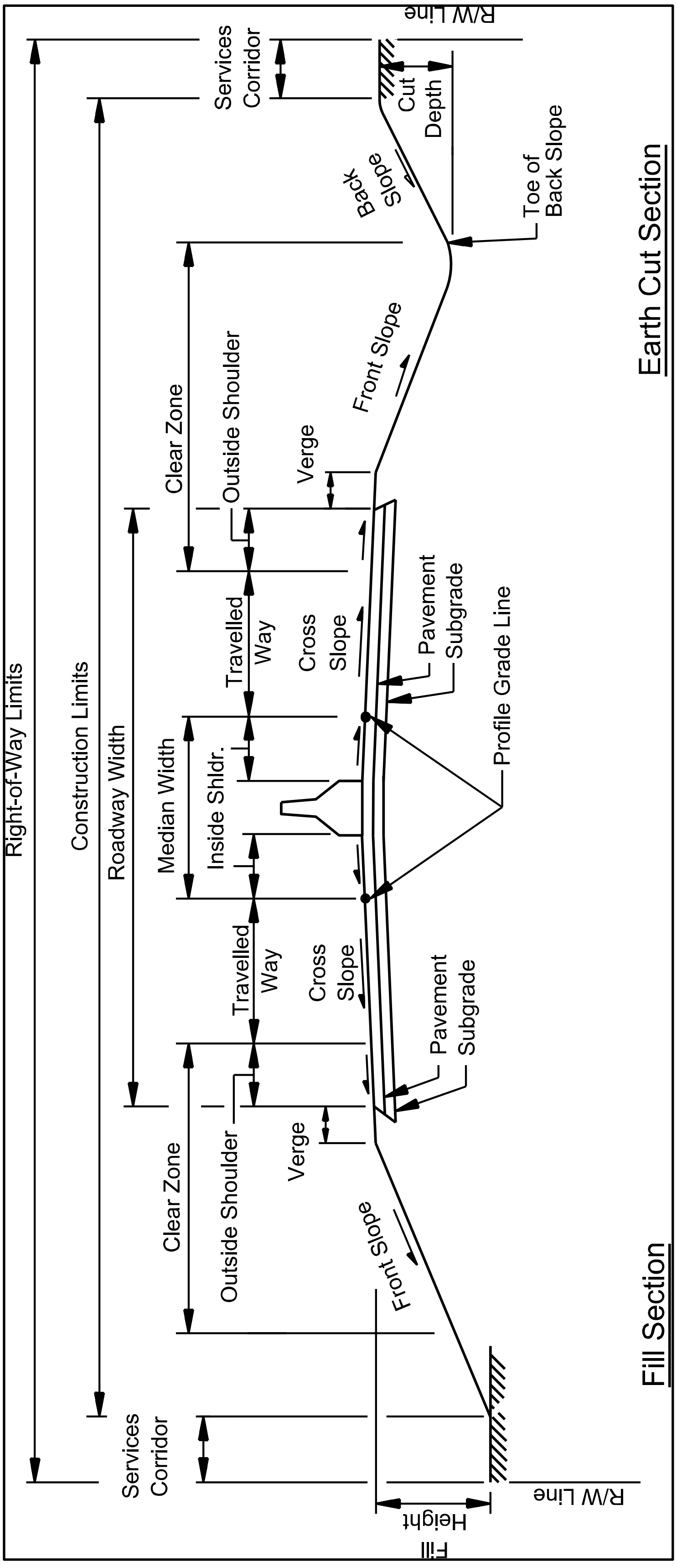


Figure 3-3: Rural Single Carriageway Nomenclature

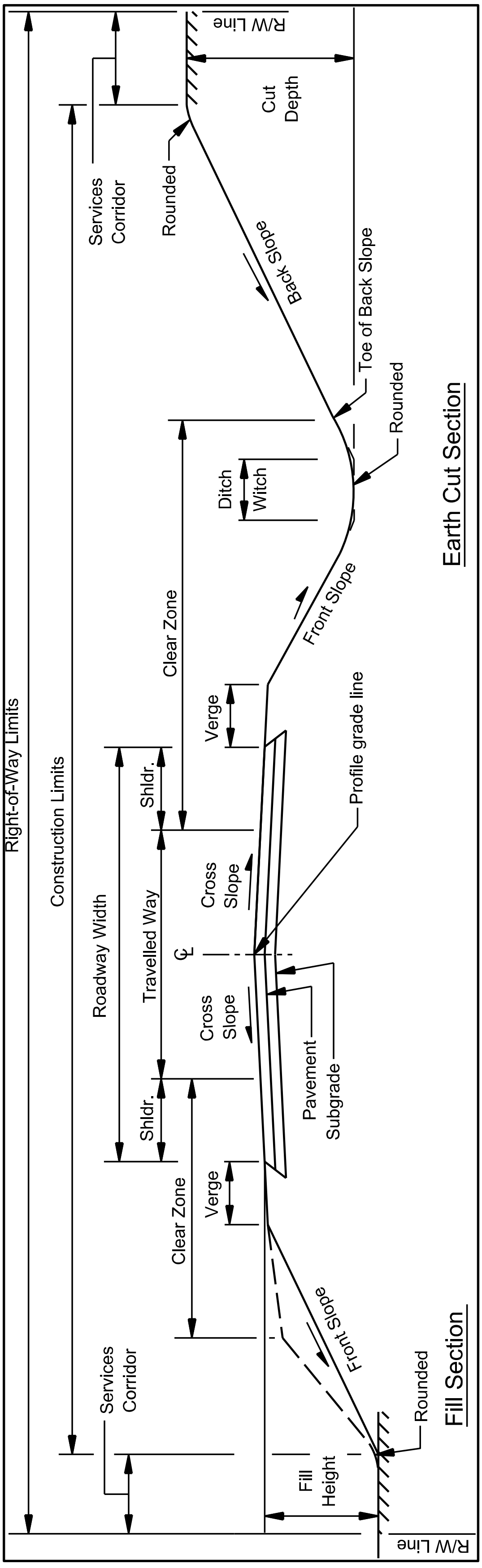
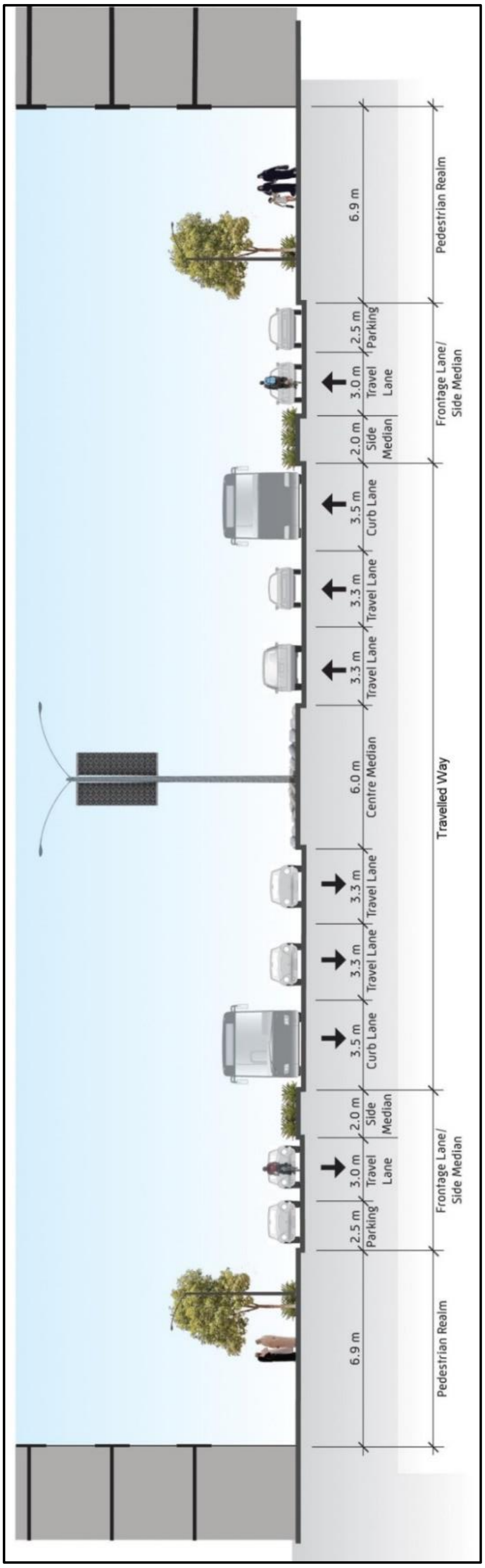
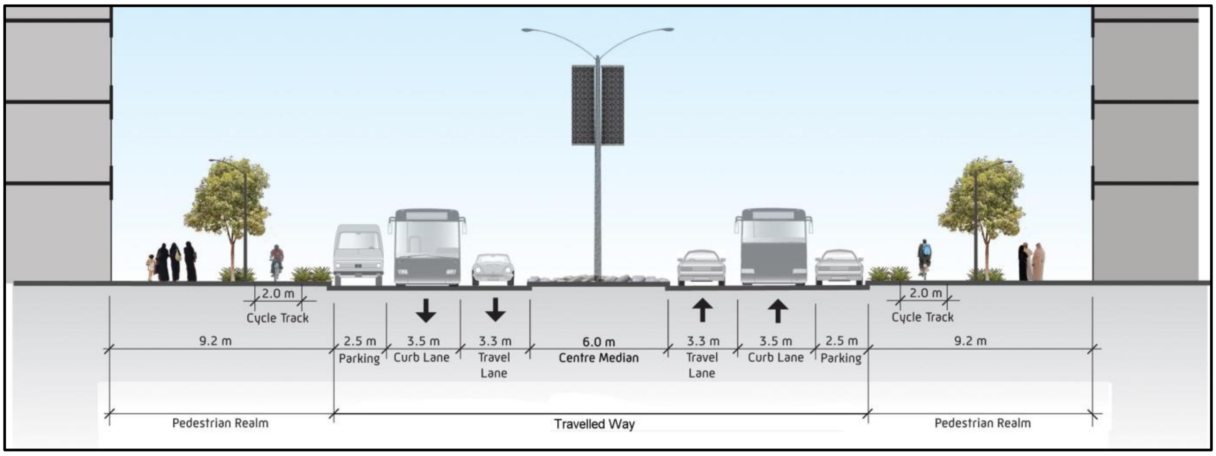


Figure 3-4: City Boulevard Nomenclature



Source: (4)

Figure 3-5: City Avenue Nomenclature



Source: (4)

- For rural local roads, travel lane widths may range from 3.0 m for lower volume roads to 3.65 m for higher volume roads.
- For rural dual carriageways and where there are a significant number of trucks, the far outside lane may be 3.75 m wide, with 4.0 m as an absolute maximum width.
- Travelled way width is one of the most important safety factors in design. A wide two-lane two-way pavement provides higher capacity, higher driver comfort levels, consistent operation and lower accident rates. The standard lane width for Boulevards and Avenues is 3.3m and the width of the first (edge) lane (for buses and larger vehicles) is 3.5m. This reduces to a standard lane width of 3.0m for Streets, Access Lanes and Frontage Lanes; although a 3.5m lane width should be maintained for designated bus routes. Storage/deceleration lane requirements associated with free-right turns shall be determined as per the right turning traffic volumes. Width of left turn lanes is 3.3m (desirable) and 3.0m (minimum).

3.2.2.2 Cross Slopes

Surface cross slopes are required for proper drainage of the travelled way on tangent sections. This reduces the hazards of wet pavements by quickly removing water from the surface and reduces the likelihood of ponding. Travel lane cross slopes are 1.5 to 2%. In addition, consider the following:

1. <u>Dual Carriageways</u>. Each roadway is crowned between the inside edge of the travelled way and inside median shoulder. See Figure 3-1. The travelled way has a uniform cross slope of 1.5 to 2.0% sloping away from the median.
2. <u>Two-Lane Single Carriageway</u>. The travelled way pavement is crowned at the centreline of the travelled way. The cross slope typically is 1.5 to 2.0% sloping away from the centreline. See Figure 3-3. However, if the ultimate development will be a dual carriageway, provide a uniform cross slope of 2% away from the proposed median as shown in Figure 3-1.
3. <u>Urban Streets and Roads</u>. For dual carriageways, the cross slope for each roadway will typical have a uniform curb-to-curb cross slope of 1.5 to 2% sloping away from the median

towards the outside of the travelled way. See Figure 3-4. For single carriageway streets, the total roadway width, from curb to curb, is typically sloped with a uniform cross slope of 1.5 to 2%. See Figure 3-5. The actual travel lane cross slope may vary depending upon the pavement surface and local practices.

3.2.3 Shoulders

A shoulder is the portion of the roadway contiguous with the travelled way that accommodates stopped vehicles, emergency use, and lateral support of subbase, base, and surface courses.

3.2.3.1 Functions

The addition of paved shoulders to the outer edge of the roadway has many advantages. The following lists some of these advantages:

- provides structural support for the travelled way;
- provides support for guardrail and prevents erosion around guardrail posts;
- prevents or minimises pavement edge drop-offs;
- increases roadway capacity;
- encourages uniform travel speeds;
- provides space for emergency and discretionary stops;
- improves roadside safety by providing more recovery area for run-off-the-road vehicles;
- provides lateral clearance for encroaching vehicles (e.g. during construction or maintenance operations);
- provides a sense of openness;
- improves sight distance around horizontal curves;
- enhances roadway aesthetics;
- facilitates maintenance operations (e.g. sand removal and storage);
- provides additional lateral clearance to roadside appurtenances (e.g. guardrail, parapet walls, traffic signals, roadway signs);
- facilitates pavement drainage, water is discharged farther from the travelled way; and
- provides space for pedestrian and cyclists use.

Shoulders shall not be applied on urban streets (other than urban freeways and expressways). Structural support is provided by the curb. Disabled vehicles can generally find a safe place to stop in driveways and side streets.

3.2.3.2 Stability

If shoulders are to function effectively, they should be sufficiently stable to support occasional vehicle loads in all kinds of weather to prevent rutting. Evidence of rutting, skidding, or vehicles being mired down, even for a brief seasonal period, may discourage and prevent the shoulder from being used as intended.

3.2.3.3 Contrast

It is desirable that the colour and texture of shoulders be different from those of the travelled way. This contrast serves to clearly define the travelled way at all times, particularly at night and during inclement weather, while discouraging the use of shoulders as additional through lanes. Bituminous shoulders all offer excellent contrast with concrete pavements. Satisfactory contrast with bituminous pavements is more difficult to achieve. Various types of stone aggregates offer good contrast. The use of edge lines as described in the Abu Dhabi *Manual on Uniform Traffic Control Device* (22) reduces the need for shoulders contrast. Edge lines should be applied where shoulder use by cyclists is expected.

3.2.3.4 Widths

Shoulder widths will vary according to the functional classification, traffic volumes, urban/rural location, and if the road is curbed or uncurbed. Note the following:

- At higher speeds, a vehicle stopped on the shoulder should clear the travelled way edge by at least 0.3 m and preferably 0.5 m. At lower speeds and traffic volumes, it may be acceptable for the stopped vehicle to encroach into the travelled way.
- For freeways and expressways, 1.2m inside shoulder and 3.0m outside shoulder shall be applied.
- For divided rural collector roads, 1.2m inside and outside shoulder shall be applied

For undivided rural local roads, shoulder is not applicable Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads,” and Chapter 9 “Urban Streets” provide additional guidance on the appropriate shoulder widths. Where cyclists will be accommodated, see the Abu Dhabi *Walking and Cycling Master Plan* (23) for guidance.

3.2.3.5 Cross Slopes

The cross slope of a shoulder depends on the type of shoulder and its application. The following summarises typical practices:

1. <u>Outside Shoulders</u>. Full-width paved shoulder on tangent sections is typically sloped at 2% and in the same direction as the adjacent travel lanes. Desirably, the shoulder cross slope should be 1% to 2% steeper than the adjacent travel lanes.
2. <u>Inside (Depressed Median) Shoulders</u>. The full-width paved shoulder on tangent sections is sloped at 2% away from the adjacent travel lanes towards the median.
3. <u>Inside (Flush Median with CMB) Shoulders</u>. The full-width paved inside shoulder on tangent sections should be sloped away from the CMB at a rate of 2%.
4. <u>Aggregate Shoulders</u>. Aggregate shoulders are sloped from 4% to 6%.

3.2.4 Rumble Strips

A rumble strip is a longitudinal design feature installed on a paved roadway surface in or near the travel lane. It is made of a series of indented or raised elements intended to alert inattentive drivers, through vibration and sound, that their vehicles have left the travel lane. On dual carriageways, they are typically installed on the median side of the roadway, as well as on the outside (right) shoulder. Figure 3-6 illustrates the common rumble strips types.

The following sections describe the common rump strip types. For additional information on the placement of rumble strips, see NCHRP Report 641 *Guidance for the Design and Application of*

Shoulders and Centerline Rumble Strips (24) and the US Federal Highway Administration (FHWA) Safety website (25).

3.2.4.1 Shoulder Rumble Strips

Shoulder rumble strips are an effective way to reduce run-off-the-road crashes and to alert motorists that they have drifted off the pavement. The greatest benefit is from the installation of continuously milled rumble strips on freeways and other facilities in which a significant number of run-off-the-road crashes occur. See Figure 3-6(a) for an illustration of shoulder rumble strips. Figure 3-7 illustrates typical placement of shoulder rumble strips.

Shoulder rumble strips can present difficulties for cyclists and pedestrians. On facilities where cyclists are permitted, special design considerations will be needed if the available clear paved shoulder width is less than 1.8 m. This applies to the outside shoulders only. See Chapter 13 “Pedestrian and Cyclists Facilities” for additional guidance on cyclists.

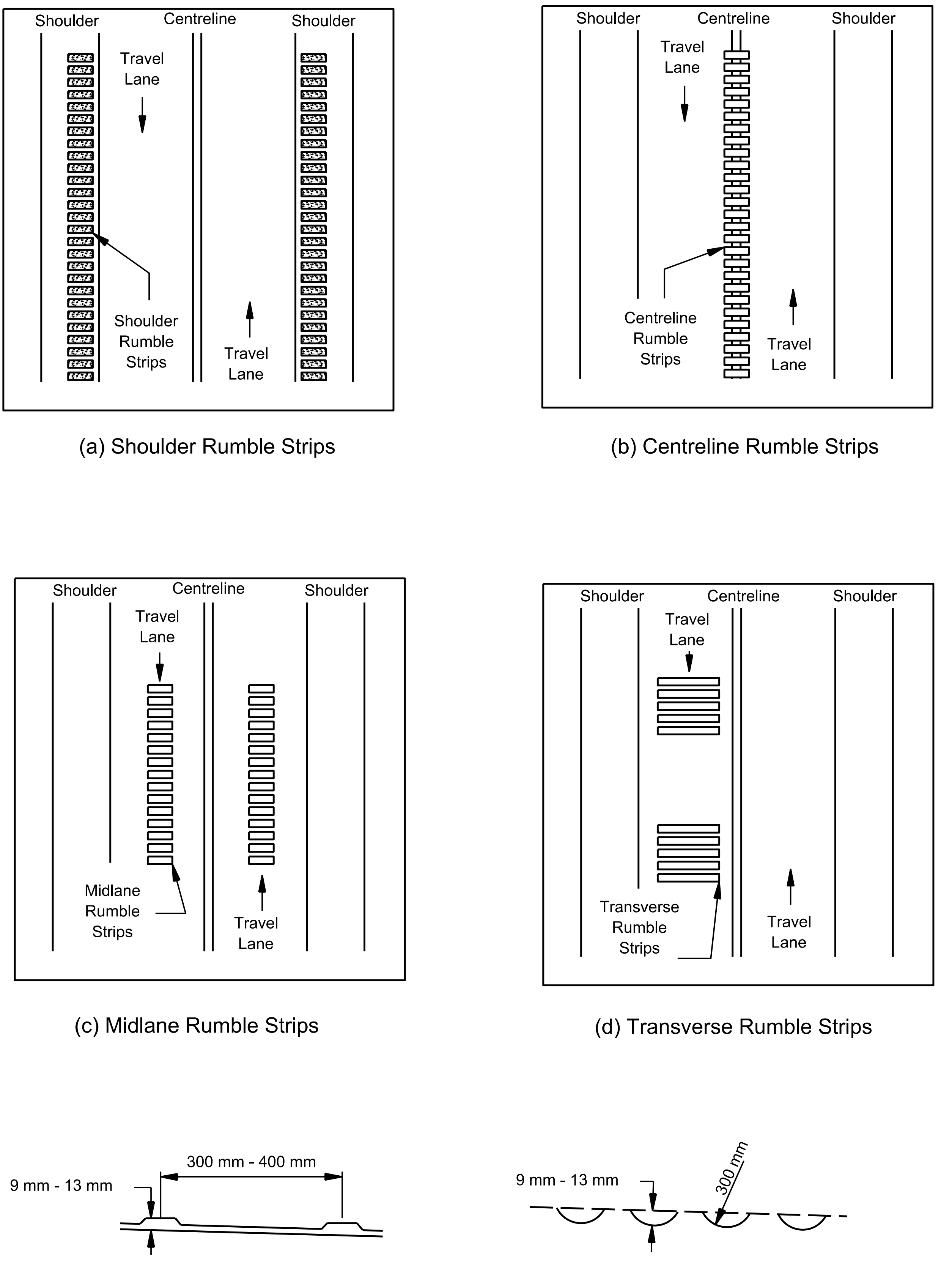
Consider installing shoulder rumble strips:

- on roadways and freeways with posted speeds greater than 80 km/h,
- along roadways that have high run-off-the-road crashes and have an adequate paved shoulder width, and
- other locations where run-off-the-road crashes may be a concern.

3.2.4.2 Centreline Rumble Strips

The primary purpose of centreline rumble strips is to warn drivers whose vehicles are crossing the centreline of two-way undivided roadways (single carriageway) to avoid potential crashes with opposing traffic. Centreline rumble strips address the problem of drowsy or inattentive drivers drifting left out of their lane and striking an oncoming vehicle. Two types of crashes are generally considered correctable by centreline rumble strips — head-on and opposite-direction sideswipes often referred to as crossover or cross-centreline crashes. See Figure 3-6(b) for an illustration of centreline rumble strips.

Figure 3-6: Rumble Strip Types



(a) Shoulder Rumble Strips

(b) Centreline Rumble Strips

(c) Midlane Rumble Strips

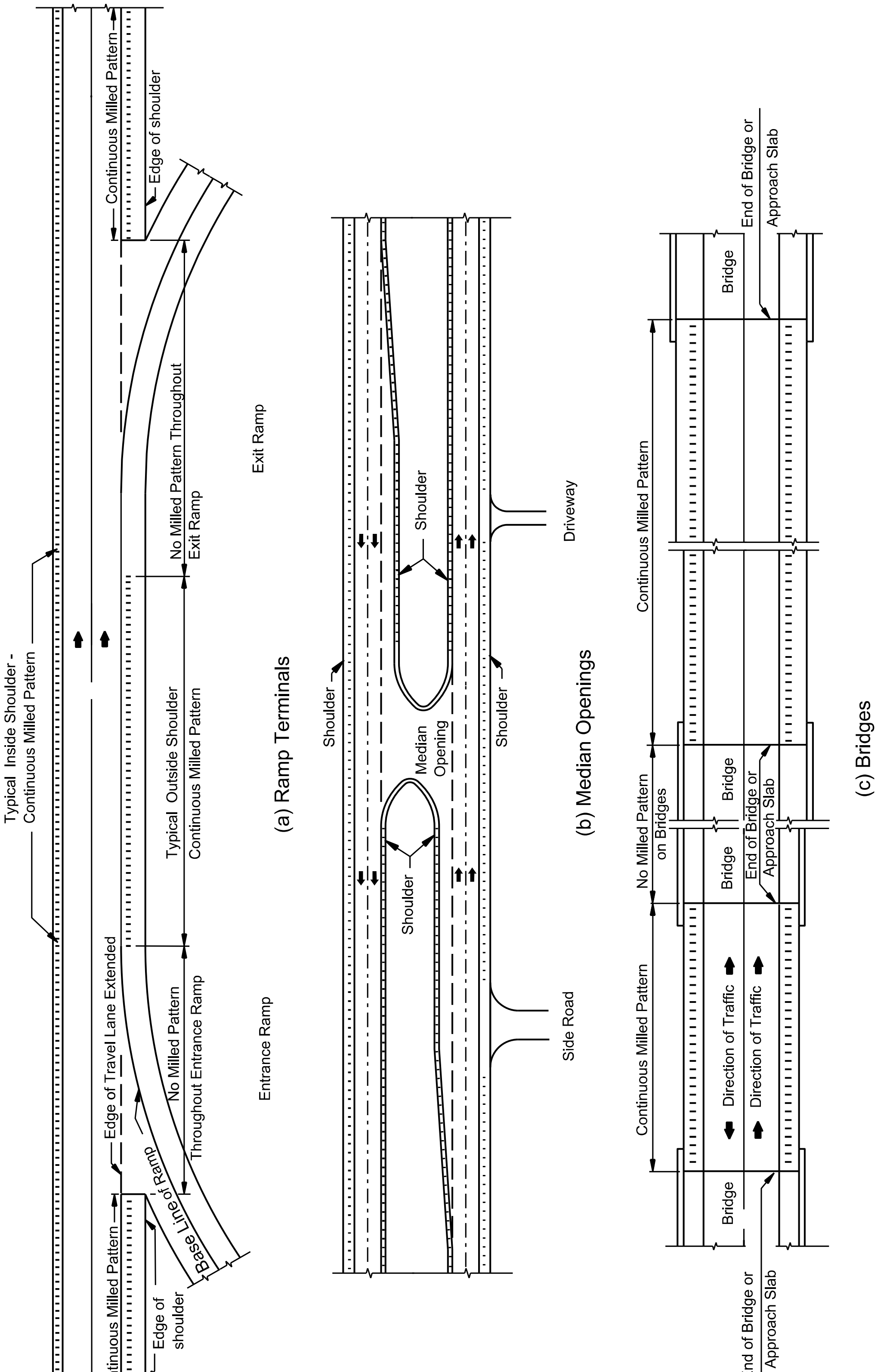
(d) Transverse Rumble Strips

(e) Raised

(f) Milled

Source: (24)

Figure 3-7: Shoulder Rumble Strip Locations



Source: (25)

3.2.4.3 Mid-Lane Rumble Strip

Mid-lane rumble strips are placed in the centre of the travelled lane. See Figure 3-6(c). Mid-lane rumble strips have the potential to mitigate both single vehicle run-off-road and crossover type crashes. Mid-lane rumble strips should only be used along roads with narrow or non-existing shoulders.

3.2.4.4 Transverse Rumble Strips

Transverse rumble strips are placed across the full width of the travel lanes. See Figure 3-6(d). Their primary purpose is to alert motorists of approaching intersections, horizontal curves, work zones or any other unexpected conditions.

It should be noted that rumble strips have minimal effect on speed reduction and are more of an alerting feature than a speed reduction feature.

Transverse rumble strip may be unsuitable in some urban areas, especially in residential neighbourhoods, as the noise that is generated when a vehicle travels over them can be a nuisance. However they are recommended in other areas (e.g. near schools, mosques, community facilities) where a large number of pedestrians congregate. Use of this form of traffic calming will need to be discussed with the Client before it is incorporated into designs.

3.2.5 Auxiliary Lanes

Auxiliary lanes are lanes adjacent to the basic through travelled way. They are intended for use by vehicular traffic for specific functions. Auxiliary lanes include:

- single left- and right-turn lanes at intersections,
- double left-turn lanes at intersections,
- truck-climbing lanes,
- acceleration/deceleration lanes at interchanges or intersections,
- weaving lanes within an interchange,
- continuous auxiliary lanes between two closely spaced interchanges,
- passing lanes, and
- bus queue-jump lanes for roads with bus priority.

Desirably, auxiliary lanes should be the same width as the adjacent through lanes. However, in many cases a greater or lesser width may be appropriate based on the site. Normally, the rate of cross slope for an auxiliary lane will be same as the adjacent through lane.

3.2.6 Curbs (Kerbs)

Curbs (also referred to as kerbs) are used on urban facilities to control drainage, delineate pavement edges, channelize vehicular movements, manage access, provide separation between vehicles and pedestrians, and present an attractive appearance. It is desirable to provide a curb offset between the face of the curb and edge of the travelled way of 0.5 m on urban streets which could help for drainage and safety. This may be especially required for application in certain cases such as turn locations, etc. where it is required based on swept path analysis.

In rural areas, curbing may be applicable where restricted right-of-way prohibits the use of a ditch section or to channelize traffic at isolated intersections. Curbs in rural areas should only be placed on the outside edge of the shoulder.

As highlighted in TR-518, kerbing has been shown to be a major contributor to the vaulting and destabilization of impacting errant vehicles, particularly at high speeds and with higher kerbs.

3.2.6.1 Warrants

Selecting a curbed section or section with shoulders and outside ditches, depends upon many variables, including vehicular speeds, urban/rural location, drainage, blowing sand, and construction costs. The following discusses some of the factors the designer should consider when determining whether a curbed section is warranted:

1. Urban Location. Because of restricted right-of-way, the need to control drainage and other constraints, curbs are usually provided on the outside edges of the roadway in urban areas.
2. Rural Location. In general, the use of curbs along rural high-speed roadways is limited to the outside edge of shoulders and to the following special conditions:
    - for roadway delineation in conjunction with channelization at intersections;
    - where there is sufficient development along the roadway and there is a need to channelize traffic into and out of properties;
    - where drainage control is required;
    - where right-of-way is restricted for roadside ditches; and/or
    - at other sites where it is deemed necessary (e.g. interchange crossroads, major intersections with restricted sight distance, where the route turns, offset left-turn lanes).

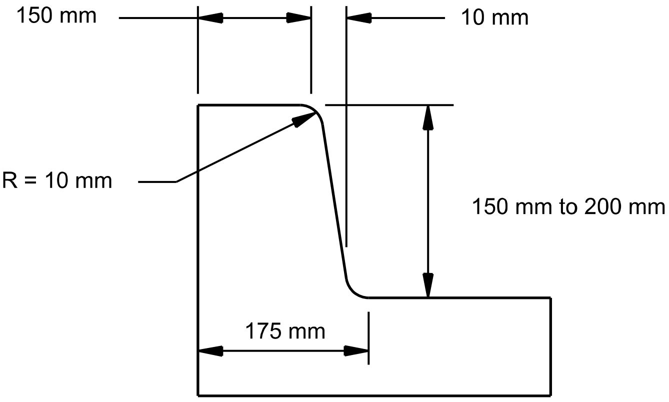
3.2.6.2 Curb Types and Uses

Vertical curbs are vertical or nearly vertical and typically have a height of 150 mm or 200 mm above the pavement surface.

Figure 3-8 illustrate typical vertical curb designs. Vertical curbs are used to discourage vehicle crossing over the curbs. Sloping curbs are those having a sloping traffic face of 150 mm or less in height. Sloping curbs, especially those with heights of 100 mm or less, can be readily traversed by a motorist when necessary. Curb heights greater than 100 mm, whether sloping or vertical, may drag the underside of some vehicles.

In general, curbs are not desirable along high-speed roadways (i.e., 90 km/h or greater). If curbs are used on high-speed roadways, only use a sloping curb and place it at the outside edge of the shoulder. For low-speed facilities, either a vertical or sloping curb can be used.

Figure 3-8: Vertical Curb Design



In urban areas, kerbs will be provided along all edges of pavement. The reasons for providing kerbs include the following:

- Provide proper drainage.
- Channelization, delineation, control of access, or improving traffic flow and safety.
- Protect pedestrians and provide continuity at ramp connections with local roads.
- Replace existing kerbs.
- Protect the roadway fence on frontage roads, where required.

Kerb Upstand – Height of the typical upstand kerb (Type-B) to be 100 mm for boulevards, avenues, streets, access lanes and frontage lanes

Kerb Painting – Kerb painting in Abu Dhabi varies dependent upon the area and denotes parking classifications and restrictions. The following colours are used:

Grey :	Standard kerbs. New kerbing is not to be painted grey.
Blue/black stripe:	Parking area (standard parking charges apply).
Blue/white stripe:	Parking area (premium parking charges apply).
Yellow/grey stripe:	No parking (to be applied at sectors where paid parking is operated).
Yellow/ grey Stripe	Visibility marking (junctions, entry/exits). Intersections, roundabouts and traffic separation islands.
Red:	Parking prohibited (adjacent to fire hydrants).
Blue:	Parking for disabled persons.

Precast kerb types and uses are shown in Figure 3-9 (Urban) and also in the Standard Drawings and are discussed below.

Kerb Types A - This 50mm upstand kerb was commonly used on Abu Dhabi streets, access lanes and frontage lanes from 2016 till the directives issued in 2018. Details are retained for information in case of maintenance works involving existing kerbs at this height.

Kerb Type-B1 – This 150mm high upstand kerb was commonly used in Abu Dhabi prior to directives issued in 2016. Details are retained for information in case of maintenance works involving existing kerbs at this height.

Kerb Type C - This kerb is used between pedestrian pavers and green areas.

Kerb Type D – This flush kerb is used between the road pavement and pedestrian ramps.

Kerb Type E - This kerb is flush with the pedestrian pavement and used as an interface between different types of pedestrian pavers.

Kerb Type F - This flush kerb is used between asphalt pavement and interlocking vehicular pavement.

Kerb Types G, H and I - These high-upstand kerbs were commonly used in Abu Dhabi prior to directives issued in 2007. Details are retained for information in the case of maintenance works involving existing kerbs.

Tyre-Friendly Kerb - This kerb is used at the interface between bus stop layby and the adjacent sidewalk.

Rolled Kerb (Mountable Kerb) - This 100mm or 50mm high dropped roll kerb may be used as an option for (1x1) streets in Residential Context areas with low operating speeds, low traffic volumes, where a proper edge zone is included to provide protective elements such as bollards, planting, planters and other street furniture”

3.2.6.3 Kerb Parameters

Placement - Kerbs should be positioned to provide the same unobstructed roadway width that is normally provided. All roadway width dimensions are measured to the front face of the kerb.

Transitions - A transition from one kerb type to another shall be done over a length of between 1.2m and 1.8m. At kerb termini, the kerb should transition from normal kerb height to zero in 3.0m.

3.2.6.4 Design Consideration

The use of a curbed section requires the consideration and implementation of many design elements. The following discusses these design considerations:

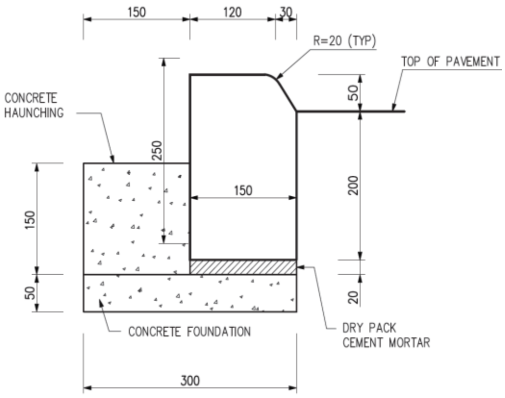
1. <u>Drainage</u>. A closed drainage system is typically used with curbed sections. The hydraulic analysis will, among other factors, depend on several curb characteristics. These include type of material (concrete, granite, or asphalt), cross slopes leading up to the curb, and shape of the curb face. In addition, it may be desirable or necessary to prevent the gutter flow from overtopping the curb. This will affect the selected curb

height. See the Abu Dhabi *Road Drainage Manual* (26) for specific criteria and procedures for drainage.

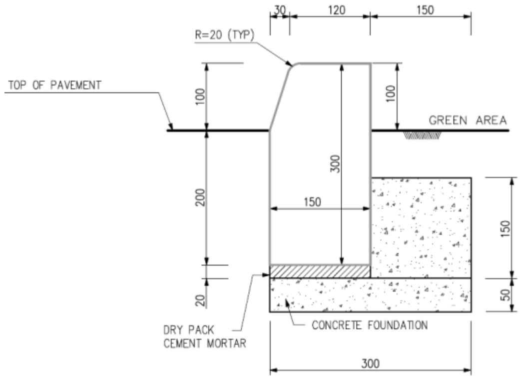
2. Curb Heights. The designer should consider the following:

- On urban boulevards, avenues, streets, access lanes, frontage lanes and parking areas, 100 mm high upstand curb is typically used
- Curb heights of 200 mm are not recommended in urban areas.
- Curb heights of 300 mm or greater may be used on rural roads to discourage drivers from crossing over the curb. For urban streets, curb heights of 300mm or more are to preferably be avoided on roads with 80kph posted speed limit (further reference is in Section 9.3.6.3).

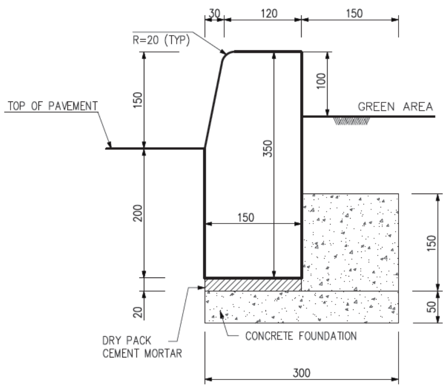
Figure 3-9 (Urban): Typical Curb Types



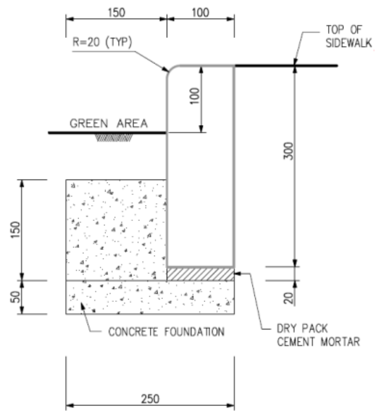
KERB TYPE A



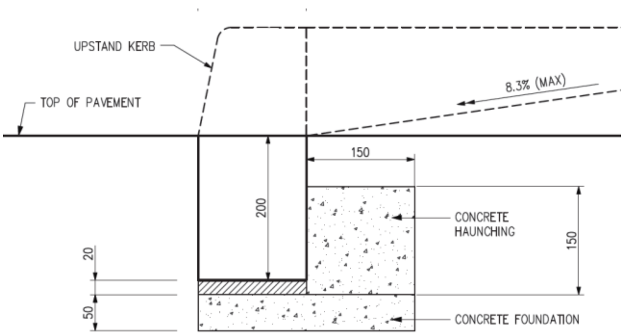
KERB TYPE B



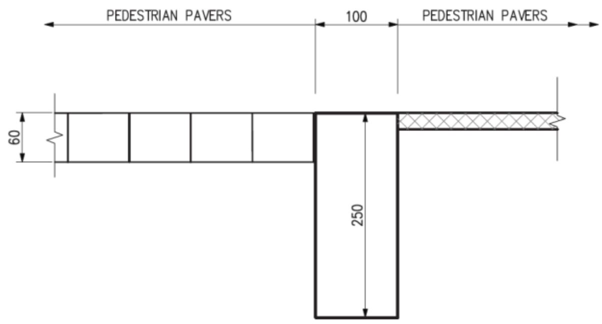
KERB TYPE B1



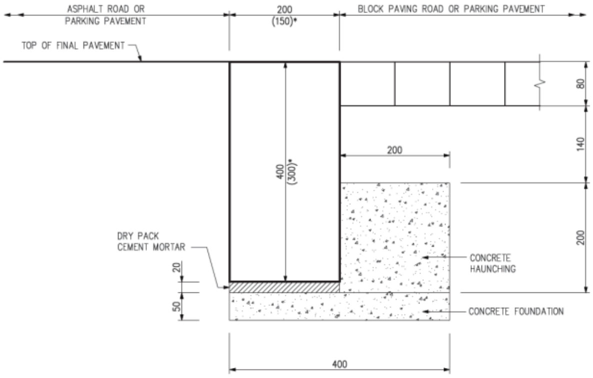
KERB TYPE C



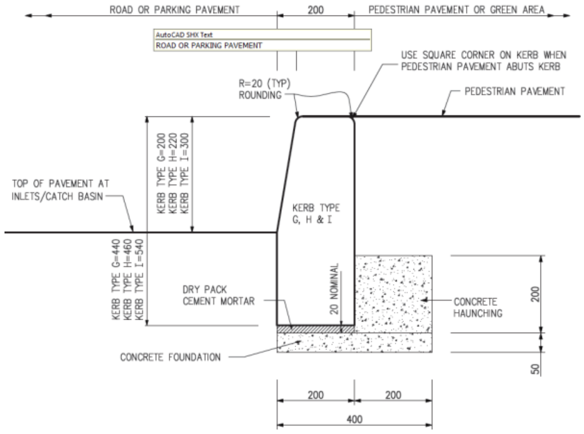
FLUSH KERB TYPE D



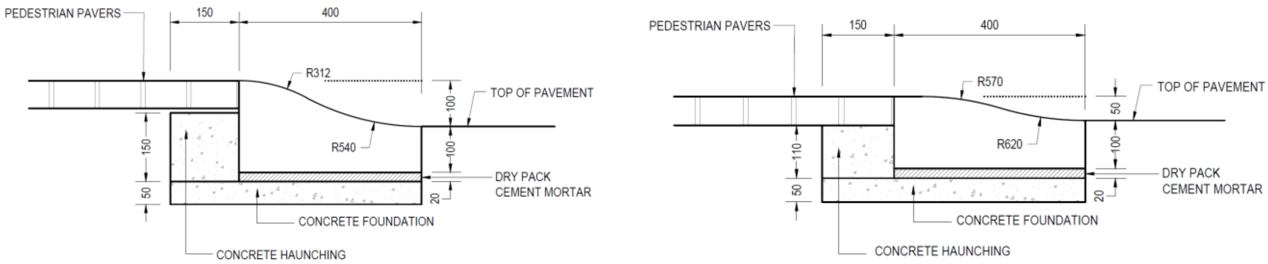
FLUSH KERB TYPE E



FLUSH KERB TYPE F



PRECAST CONCRETE KERB TYPES G, H & I
(FOR THE MAINTENANCE OF OLD KERB TYPES A, B & C)



ROLLED KERB

All dimensions are in millimetres unless otherwise noted.

Source: (27)

3. Roadside Safety. Curbs may pose a roadside hazard because of their potential to adversely affect a run-off-the-road vehicle. When evaluating curbs relative to roadside safety, the designer should consider the following:

   a. Design Speed. Facilities in rural areas with design speeds greater than 70 km/h should be designed without curbs. However, if necessary, a sloping curb may be used on high-speed facilities. Facilities with a design speed of 70 km/h or less may use either a sloping or a vertical curb. For urban streets, reference is made to Section 9.3.6.3.

   b. Traffic Barriers. The use of curbs with a traffic barrier is discouraged and, specifically, curbs higher than 100 mm should not be used with a barrier.

   c. Redirection. Curbs offer no safety benefits on high-speed roadways and should not be used to redirect errant vehicles.

4. Future Resurfacing. The designer should consider the likelihood and depth of a future resurfacing course when determining the initial curb height. For example, the curb height may be determined by the sum of the water overtopping depth (based on a drainage analysis) and the future resurfacing depth.

5. Blowing Sand. Curbs should not be used in areas where blowing sand is a concern; sand will accumulate next to the curb. Where curbs are necessary, only use a 100-mm high curb. For guidance on blowing sand, see Section 3.5.

6. Universal Access. Curbs should be designed with curb ramps at all pedestrian crosswalks to provide adequate access for the safe and convenient movement of physically disabled individuals. See the Abu Dhabi *Walking and Cycling Master Plan* (23) for the design and location of curb ramps.

3.3 Medians

A median is defined as the portion of a cross section that separates the opposing directions of the travelled ways.

3.3.1 Functions

The principal functions of a median are to:

- provide a separation between opposing travelled ways;
- provide a recovery area for out-of-control vehicles;
- provide a storage area for emergencies;
- manage access to the roadway;
- prevent undesirable turning movements, provided the median is non-traversable;
- provide areas for deceleration and storage of left-turning and U-turning vehicles;
- provide a location for offset left-turn lanes to minimise sight restrictions caused by opposing left-turn vehicles;
- minimise headlight glare;
- provide an area for storage of vehicles crossing the mainline roadway at intersections;
- facilitate drainage and sand collection (raised or depressed medians);
- permit the use of shorter span lengths for overhead structures;
- provide an area for pedestrian refuge (raised median); and
- provide width for future expansion of the roadway.

3.3.2 Median Widths

In general, the median should be as wide as can be used advantageously. The median width is measured from the inside edge of the two travelled ways and includes both inside shoulders and/or median curb. Medians may range in width from as little as 1.2 m (rural) or 2.0 m (urban) to 20 m or more, with street lighting, drainage, and landscaping. Table **3-1** presents typical median widths based on the road classification.

Table 3-2 provides minimum median widths based on the desired function of the median. The actual design width will depend on the functional class of the road, design speed, type of access management proposed, available right-of-way, construction costs, maintenance considerations, acceptable median slopes, anticipated ultimate development of the facility, operations at crossroad intersections, and field conditions. When selecting the median width, the designer should consider the following:

1. Unsignalised Intersections: Curbed median width on urban streets varies between 2m to 6m; see Table 3.1 and USDM for median widths as per the street type. In rural areas and where trucks and/or buses are commonly expected to stop in the median (e.g. truck terminals, light industrial), depressed median widths of 18 m to 20 m are recommended to allow trucks to stop between the roadways.
2. Signalised Intersections: Wide medians may lead to increased clearance times and inefficient traffic operations at signalised intersections.
3. Left-Turn Lanes: On Boulevards and Avenues, provide a minimum 2.0-m pedestrian refuge in the median when introducing left-turn lanes.
4. Median Barriers: With narrow medians, a median barrier may be warranted. See the Abu Dhabi *Roadside Design Guide* (28). Desirably, the designer should select a median width that will be wide enough to eliminate the need for a barrier.
5. Operations: Several vehicular manoeuvres at intersections are partially dependent on the median width of the mainline highway (e.g. U-turns, left turns). The designer needs to evaluate these likely manoeuvres at intersections and provide a median width that will accommodate the selected design vehicle. Also, consider the need for single or dual left-turn lanes. For more information on designing left-turn lanes at intersections, see Chapter 10 "Intersections."
6. Uniformity: In general, a uniform median width is desirable. However, variable-width medians may be advantageous where right-of-way is restricted, at-grade intersections are widely spaced (1 km or more), and/or where independent alignments are practical.
7. Sight Distance: Where a median barrier is proposed at a horizontal curve, the median width may be a factor in whether or not adequate sight distance is available around the horizontal curve. Section 5.5 discusses horizontal sight distance.
8. Separation: Median widths of 12 m or greater, from the driver's perspective, are considered to be physically and psychologically separated from the opposing traffic. Median widths of 18 m or more can be landscaped without restricting the roadside recovery area.
9. Maintenance: If glare screens, light poles or other appurtenances are placed on a median barrier, the desirable median width is 8 m. This provides sufficient clearance for emergency or maintenance vehicles to stop on the shoulders without blocking the travelled way.
10. Zone of Intrusion: Median width must take into account the ZoI of the RRS/VRS to be used and ensure that the street lighting and foundations are not within this ZoI and at risk of being struck (also refer to TR-518)

Table 3-1: Median Widths

	Urban	**Rural**
Collectors	Minimum 2.0 m Typical 6.0 m	Minimum 4.0 m Typical 6.0 m
Minor Arterials	Minimum 5.0 m Typical 6.0 m	Minimum 5.0 m Typical 8.0 m to 12.0 m(2)
Principal Arterials	Minimum 5.0 m Typical 6.0 m	Minimum 6.0 m Typical 8.0 m to 15 m(2)
Expressways	Minimum 8.0 m(3)	Minimum 10.0 m(3)
Freeways	Minimum 10.0 m(4) to 20 m	Minimum 10.0 m(4) to 20 m

Notes:

11. *1. Narrower widths may be appropriate at signalised intersections. See*

12.

13.

14.

15.

Table 3-2*.*

2. *Typical for future lane widening.*
3. *May need to provide a median barrier; see the Abu Dhabi Roadside Design Guide* (28)*.*
4. *Median widths may be increased to suit visibility requirements.*

Table 3-2: Minimum Median Widths Based On Function

	At Signalised Intersections	**Elsewhere**
Minimum to accommodate signal heads	1.8 m	1.8 m
Minimum (curbed) to separate traffic safely	1.2 m	1.2 m
Minimum to accommodate pedestrians safely	2.0 m	3.5 m
Minimum to provide left-turn lane	4.2 m	4.75 m
Minimum to provide U-turn	See Chapter 10 “Intersections”	5.05 m

Desirable for U-turn	See Chapter 10 "Intersections"	10.75 m or more
Minimum for provision of effective landscaping	n/a	8.0 m

n/a: not applicable – other considerations govern
Note: In urban areas, the minimum median width is 2.0 m.

3.3.3 Median Types

Figure 3-10 provides typical sections for various median types — flush with concrete barrier, raised, and depressed.

3.3.3.1 Flush Medians

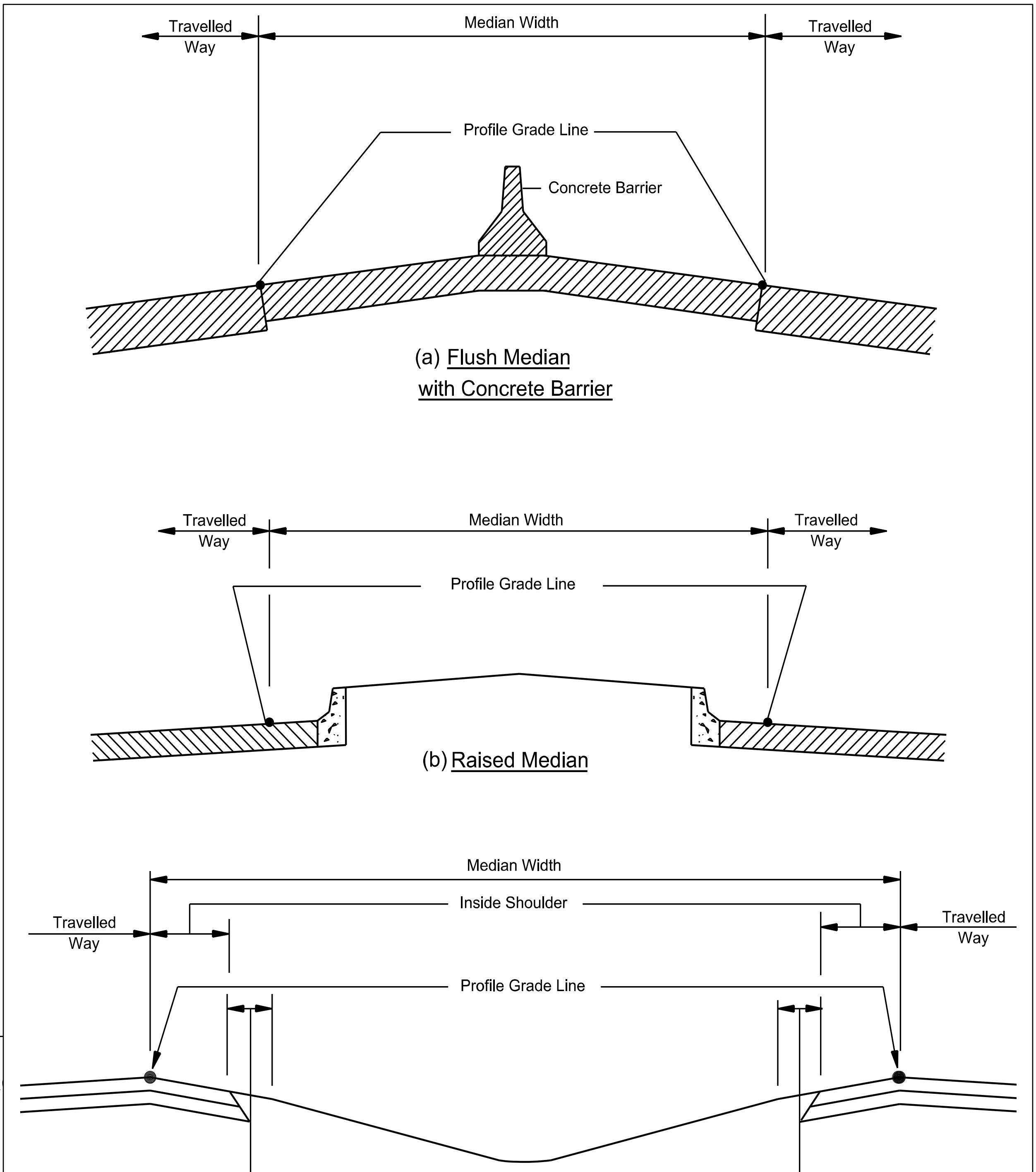
A flush median is defined where the surface is constructed as a smooth plane in conjunction with the adjacent roadway pavement. Flush medians may be used on low-speed urban roadways and streets where left-turn lanes are common and where there is a need to provide some separation between opposing traffic. However, they do not offer a pedestrian refuge in the middle of the roadway. Flush median should be crowned in the centre to avoid water ponding in the median area. With high-speed roadways, flush medians can also be used with the placement of concrete barriers; see Figure 3-10(a). In this case, the barrier is placed at the crown of the roadway and water is drained to the outside.

The following discusses typical widths for flush medians:

1. Flush. Widths for a flush median on an urban street can range from 1.2 m to 4.8 m. To accommodate a separate left-turn lane, a flush median should desirably be 4.0 m wide. Either this will allow for a 3.3 m or 3.65 m turn lane and a minimum separation of 700 mm or 350 mm, respectively, between left-turning vehicles and the opposing traffic.

2. Flush with Concrete Barrier. A flush median with a concrete barrier may be used on urban freeways and expressways where the right-of-way does not allow for the use of a depressed median. The minimum width of a flush median for an urban freeway is 8 m. This allows for the use of two 3.0 m wide left shoulders and space for the width of the concrete barrier.

Figure 3-10: Typical Median Types



3.3.3.2 Raised Medians

A raised median is defined as a median if it contains a raised curb within its limits. See Figure 3-10(b) Usually, a raised median is proposed when there is a need to manage access to the street and to control left-turn movements (urban streets). When compared to flush medians, raised medians offer several advantages:

- mid-block left turns are controlled;
- left-turn channelization can be more effectively delineated;
- a distinct location is available for traffic signs, signals, pedestrian refuge and cyclists; and
- limited physical separation is available.

The disadvantages of raised medians when compared to flush medians are:

- They are more expensive to construct and more difficult to maintain.
- They may need greater widths to serve the same function (e.g. left-turn lanes at intersections) because of the raised island and offset between the curb and travel lane.
- Curbs may result in adverse vehicular behaviour upon impact.
- Prohibiting mid-block left turns may overload street intersections and may increase the number of U-turns.
- Access for emergency vehicles (e.g. fire, ambulance) may be more difficult.

If a raised median is used, the designer should consider the following in the design of the median:

1. <u>Curb Type</u>. Sloping, upstand, vertical, or jersey shaped curbs are typically used for raised medians. Sloping or drop curbs may allow vehicles to exit the travel lanes where necessary for emergency reasons. Do not use vertical or upstand curbs along high-speed roadways.
2. <u>Speed</u>. Vertical curbs may be used where the design speed is 70 km/h or less. Sloping curbs may be used at any design speed.
3. <u>Curbs at Turn Bays</u>. Where the design speed is 70 km/h or less, either vertical or sloping curbs may be used adjacent to channelized left- or right-turn lanes.
4. <u>Curb Types with Appurtenances</u>. If practical, the placement of appurtenances within the raised median is discouraged. However, if appurtenances (e.g. traffic signal poles, light poles, signs, bridge piers) are required in the median and the median width is less than 3.6 m, a 150-mm vertical curb may be necessary on low-speed facilities.
5. <u>Desirable Width</u>. If practical, the width of a raised median should be sufficient to allow for the development of a channelized left-turn lane. For example, a 4.85-m median width would allow for a 3.65-m turn lane and a minimum 1.2-m raised island.
6. <u>Minimum Width</u>. The minimum width of a raised island should be 1.2 m.

7. Cross Slopes. If the median is curbed and paved, the median surface should be designed with slopes of 2.5%, and slope away from the centre of the median.

8. Landscaping. Where raised medians are landscaped with vegetation, due consideration needs to be given to how this will be maintained especially on 80kph roads or higher.

3.3.3.3 Depressed Medians

Wherever practical, the designer should provide a depressed median on rural freeways and other dual carriageways. See Figure 3-10(c). Depressed medians have better drainage characteristics and, therefore, are preferred on high-speed roadways. In addition, they provide the driver with a greater sense of comfort and freedom of operation. Where a depressed median is proposed, the designer should consider the following:

1. Widths. Depressed medians should be as wide as practical to allow for the addition of future travel lanes on the inside while maintaining a sufficient median width. In addition, the median should be sufficiently wide so that a median barrier will not be warranted.

2. Drainage. Because water is allowed to flow into a depressed median, the designer needs to consider drainage when determining the appropriate depth of a depressed median. The median ditch should be designed so that the depth of flow during the design-year discharge (e.g. Q_{10}) will be at least 150 mm below the subgrade. See the Abu Dhabi *Road Drainage Manual* (26) for additional guidance on drainage requirements.

3. Additional Lanes. If the ultimate design will include adding lanes in the median, the minimum median width is 14 m — two 3.65-m travel lanes, two 3.0-m inside shoulders, and a 0.7-m concrete median barrier.

4. Road Safety. The adverse road safety implications should be analyzed considering risk of errant vehicle losing control and overturning or hitting rigid objects in the depressed median

3.3.4 Median Selection

When selecting a median type, recognition must be given to urban/rural location, access needs, design speeds, availability of right-of-way, safety, capacity, intersection spacing, traffic signals, sight distance, turn-lane length, economics, environmental impacts, public appearance, and functional classification. Higher functional classifications will warrant a greater effort in managing access to a street or roadway, and in retaining mobility.

The designer should consider all features of a particular route to determine the best median alternative. Do not focus on one particular feature when selecting a typical section, but rather look for a preponderance of the features that make the most suitable alternative. It is advisable to conduct a thorough site review of each project to check for site-specific traffic generators that may dictate a change in the typical section.

The designer should consider the crash data for the entire project and the driveway density on a per kilometre basis. Reviewing crash data for the entire project allows the designer to evaluate crash types and severity to determine crash patterns for the entire project.

On certain projects, there may be more than one typical section. Varying the median type may be necessary and/or desirable. The length of a project will be a major influence in this determination. On relatively short roadway sections, the number of different typical sections should be limited to the predominant median configuration. On longer roadway projects, changes to the typical section should generally be made at the borders of natural cultural subdivisions.

3.4 Roadside Elements

3.4.1 Verges

The verge is a part of the cross section that acts as a buffer zone between the edge of the pavement (curb or back-of-shoulder) and the surrounding physical features. Verges help to provide stability to the edges of the pavement construction, reducing the chances of damage from erosion. They also may accommodate roadside barriers, road signs and structures, traffic signals, and road lighting. The following provides guidance on verges:

1. <u>Rural</u>. Verges are normally unpaved in rural areas.
2. <u>Urban</u>. Verges are not required on curbed roads with sidewalks, although landscaped may be provided, if desired.
3. <u>Utilities</u>. Utilities (e.g. electricity and water) are laid underground along road corridors, and it is normal to allow for them within the roadway right-of-way. These services are usually laid in the verge, and therefore the verge may need to be significantly wider than would be required purely for traffic safety reasons.
4. <u>Width</u>. Verge widths may vary from a desirable minimum of 2.25 m (for traffic safety reasons) up to the limits of the right-of-way.
5. <u>Drainage</u>. If a paved verge is provided for drainage purposes, it should normally be designed with a 2.5% cross slope away from the roadway. However, for wider paved verges, design the verge with drainage collection points located within the verge itself.
6. <u>Sight Distance</u>. It is important to ensure that instalments in the verge (e.g. structures, signs, landscaping, bridge abutments) do not obstruct the sight distances required for the road. Isolated obstructions can normally be ignored, but massive or continuous obstructions need to be identified and appropriate measures taken to achieve the sight distance criteria. Particular care should be taken at intersections where the number of signs and other items of street furniture are greater than on the open road.

3.4.2 Side Slopes

Earth slopes are required to provide roadside and median ditches adjacent to roadway facilities, and to provide a stable transition from the roadway profile to adjacent terrain features. Flat slopes also facilitate vegetation establishment, soil stability, and blowing sand control. Flat and well-rounded side slopes, combined with proper roadway elevations above natural ground lines also enhance the roadway aesthetically, as well as provide easy accessibility with regard to maintenance operations.

Using broad flat slopes on roadside ditches, which are visible to the driver, lessen the feeling of restriction and add considerably to a driver's willingness to use the shoulder and earth slope area in emergencies. The use of flat side slopes for roadside ditches reduces both the depth and velocity of water, and thereby minimises damage from erosion.

3.4.2.1 Fill Slopes

Fill slopes are the front slopes extending outward and downward from the shoulder or verge hinge point to intersect the natural ground line. The slope criteria are dependent upon the functional classification, fill height, urban/rural location, and the presence of curbs. Although Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads,” and Chapter 9 “Urban Streets” provide design criteria for fill sections for each road classification, the designer must also consider right-of-way restrictions, utility considerations, roadside safety, and roadside development in determining the appropriate fill slope.

Desirably, fill slopes should be 1V:6H or flatter. A 1V:3H slope is a practical maximum when considering maintenance operations, erosion control, and roadside safety. Slopes steeper than 1V:3H will normally require a roadside barrier; see the Abu Dhabi *Roadside Design Guide* (28) for guidance on roadside barriers.

3.4.2.2 Ditch Section

On facilities without curbs, roadside ditches are provided adjacent to embankment locations and in cut sections to control drainage. As illustrated in Figure 3-11, the ditch section includes the front slope, ditch width, and back slope for the type of roadway. Figure 3-12 shows a typical ditch section. Where the cut height exceeds 3 m from the bottom of the ditch to the existing ground line, the designer may consider using a 1V:2H back slope beyond the clear zone to reduce excavation costs.

3.4.2.3 Material and Soil Conditions

The designer must ensure that permanent erosion control is considered in the design of ditches in cut slopes. The designer should contact the landscape architect and the geotechnical engineer who will review the existing soil conditions to determine if additional measures may be required to control erosion (e.g. special plantings, paving). It will be the road designer’s responsibility to consider their recommendations for incorporation into the plans.

3.4.2.4 Hydraulic Design

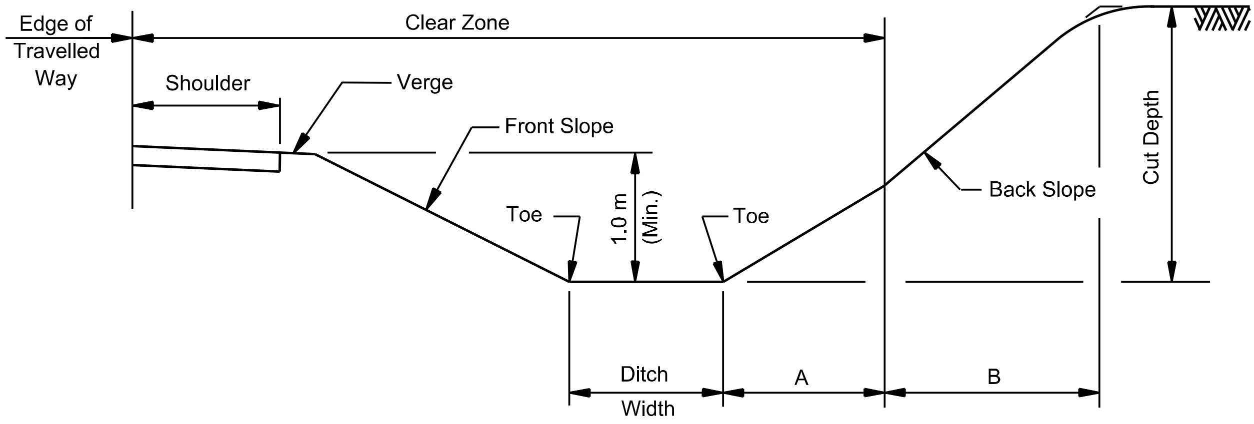
Design roadside and median ditches to ensure proper drainage of the pavement subgrade and the adequate conveyance of surface flow without creating erosion of ditch sections. See the Abu Dhabi *Road Drainage Manual* (26) for guidance on drainage requirements.

3.4.2.5 Cut Sections with Curbs

On facilities with curbs, a shelf may be provided with a back slope beyond the shelf. The shelf is usually sloped towards the roadway to eliminate the need for a separate drainage system behind the curb. Where sidewalks are present or anticipated in the future, provide a shelf width of 3.0 m with a cross slope of 2%. Where sidewalks are not present or anticipated in the future, the shelf cross slope should be 2.5%. See Figure 3-13

Although curbs are not considered fixed objects in the context of a clear zone, they may have an effect on the trajectory of an impacting vehicle and may have an effect on a driver’s ability to control a vehicle that strikes or overrides one. The magnitude of this effect is largely influenced by vehicle speed, angle of impact on the curb, curb configuration, and vehicle type. Sloping curbs with

Figure 3-11: Typical Ditch Section



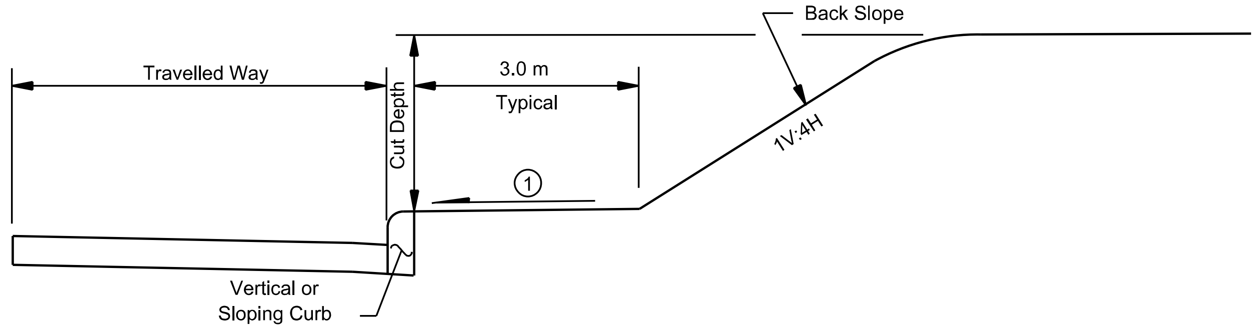
Project Scope of Work	Front Slope (V:H)	Minimum Ditch Width	Back Slopes (V:H)	
			A	B
New construction or reconstruction	1:6	1.2 m (1)	1:3	(2)
Existing to remain (Reconstruction)	1:4	500 mm (1)	1:3	(2)

Notes:

1. *Consider providing a wider ditch where detention storage of storm water is required.*
2. *Where the height of cut exceeds 3 m, consider using a 1V:2H back slope beyond the clear zone. See Abu Dhabi Roadside Design Guide* (28) *for clear zone distances.*

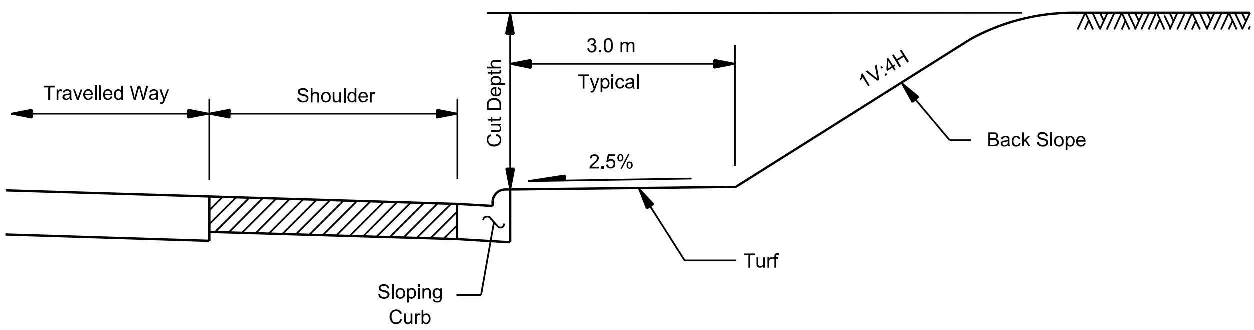
Figure 3-12: Roadway Ditch Section

Figure 3-13: Typical Cut Sections with Curbs



Notes:
1. 2% If sidewalks are present or anticipated.
2. 2.5% If sidewalks are not present or anticipated.

(a) Design Speed ≤ 70 km/h



(b) Design Speed ≥ 80 km/h

heights up to 100 mm may be considered for use on high-speed facilities where necessary due to drainage considerations, restricted right-of-way, or need for access control. When used under these circumstances, curbs should be located at the outside edge of shoulder. Vertical curbs may be used with design speeds less than 70 km/h.

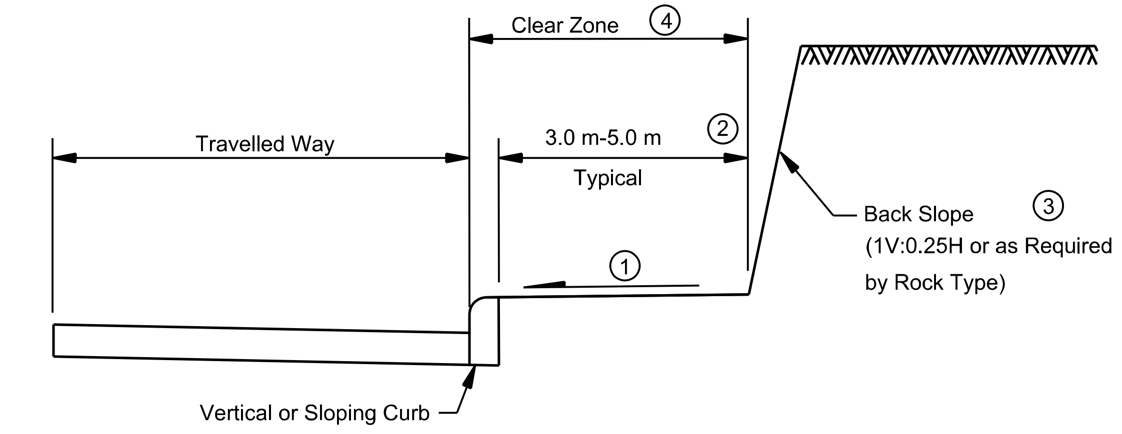
3.4.2.6 Rock Cuts

In rock cuts, the back slope generally is 1V:0.25H, but can vary depending on the type of rock and field conditions; see Figure 3-14. For large cuts, benching of the back slope may be required. For guidance on benching designs in rock cuts, the designer should contact the geotechnical engineer.

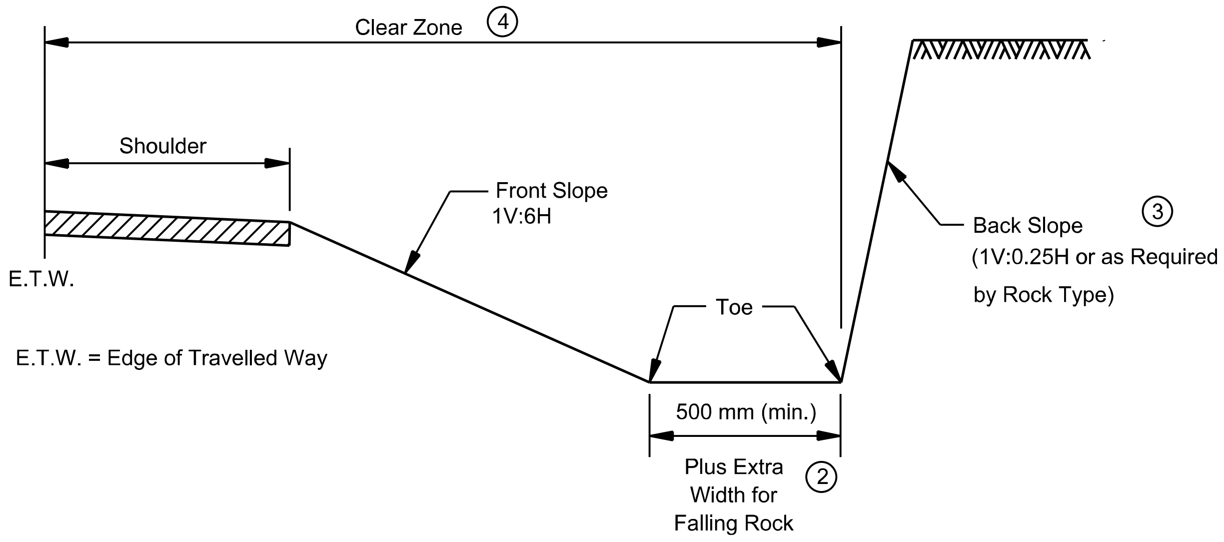
Conduct a cost-effective study to determine if a rock cut should encroach into the clear zone area or be located outside of the clear zone. If the rock face is located within the clear zone, provide a smooth rock cut or shield it with a roadside barrier. If the rock face and ditch to capture falling rock is outside of the clear zone, a roadside barrier typically will not be warranted.

In addition, conduct a hydraulic analysis to evaluate the need to control cascading water from the top of the cut and to determine the conveyance needs or roadside drainage at the toe of the cut.

Figure 3-14: Typical Rock Cut Section



(a) Design Speed ≤ 70 km/h



(b) Design Speed ≥ 80 km/h

Notes:

① *Use 2% if sidewalks are presented or anticipated; otherwise, use 2.5%.*

② *Discuss with the geotechnical engineer to determine extra width needed for falling rock.*

③ *Back slopes in rock may require benching. Contact the geotechnical engineer for guidance.*

④ *See Abu Dhabi Roadside Design Guide* (28) *for clear zone discussion.*

3.4.2.7 Geotechnical Features

During the project development stage, the designer must ensure that the topography and geology of the selected alignment and profile are compatible with the proposed fill and cut slope sections. The use of earthwork computer programs will help to determine cross section limits. In addition, the Geotechnical Report and any additional data must be analysed during the preparation of the construction plans. This will help to ensure the stability of selected cut and fill slopes that are 1V:3H or steeper.

Major or unusual geotechnical features within the project limits will require special consideration. These features should be reviewed with the geotechnical engineer.

3.4.2.8 Aesthetics

The designer should explore various options to improve the visual impact a roadway will have on the landscape. Varying the cross section elements will typically improve the aesthetics of the roadway. This may include:

- increasing or decreasing the side slopes to reduce the magnitude of exposed cut and fill slopes,
- reducing ditch widths or depths to reduce the amount of cut,
- using slope rounding to blend cuts and fills into the natural ground,
- warping side slopes to match the natural landscape,
- retaining existing vegetation,
- using a raised or depressed median with plantings, and
- providing structures that match the natural landscape.

3.4.2.9 Drainage Channels

The primary function of drainage channels, which parallel the roadway, is to collect surface runoff from the roadway and adjacent areas and convey the accumulated runoff to acceptable outlet points. The designer should design the channel to convey the design runoff and to accommodate excessive storm water with minimal highway flooding or damage. See the Abu Dhabi *Road Drainage Manual* (26) for guidance.

Roadside safety is also important in the design of drainage channels. Recoverable slopes are those on which a motorist can retain or regain control of a vehicle. Slopes flatter than 1V:4H are considered recoverable. Smooth slopes with no significant discontinuities and no protruding fixed objects are desirable from a safety standpoint. The designer should locate drainage channel cross sections that are not considered traversable at or beyond the clear-zone distance. See the Abu Dhabi *Roadside Design Guide* (28) for guidance.

3.4.3 Sidewalks/Pathways

Generally, sidewalks are an integral part of city streets are provided on both sides of the roadway. However, sidewalks in rural areas are still often justified at points of community development (e.g. schools, local businesses, shopping centres, industrial plants) that may result in pedestrian concentrations along the roadway. If pedestrian activity is anticipated, the designer should review the sidewalk warrants and design criteria presented in the Abu Dhabi *Walking and Cycling Master Plan* (23).

3.4.4 Cyclists

Provisions for cyclists are an important consideration where new roadways are being constructed or existing roadways are being widened or otherwise improved. This is particularly true in urban areas and where tourism is a major factor. In many instances, minimal capital investment measures can considerably enhance safety and provide capacity for cyclist traffic. See Abu Dhabi *Walking and Cycling Master Plan* (23) for guidance on cycling facilities.

3.4.5 Clear Zones/Clearances

The term "clear zone" is used to designate the unobstructed, relatively flat area provided beyond the edge of the travelled way for the recovery of errant vehicles. The clear zone includes any shoulder or auxiliary lanes. The clear zone should be clear of all unyielding objects (e.g. trees, sign supports, utility poles) and any other fixed objects. A minimum clearance of 0.5 m should be provided between the face of curb and obstructions (e.g. utility poles, fire hydrants) on all streets. See the Abu Dhabi *Roadside Design Guide* (28) for guidance on clear zones and horizontal clearances.

3.5 Traffic Barriers

Traffic barriers are used to prevent vehicles that leave the travelled way from colliding with objects that have greater crash severity potential than the barrier itself. Because barriers are themselves a source of crash potential, the designer needs to carefully consider their use. For in-depth guidance regarding traffic barriers, see the Abu Dhabi *Roadside Design Guide* (28).

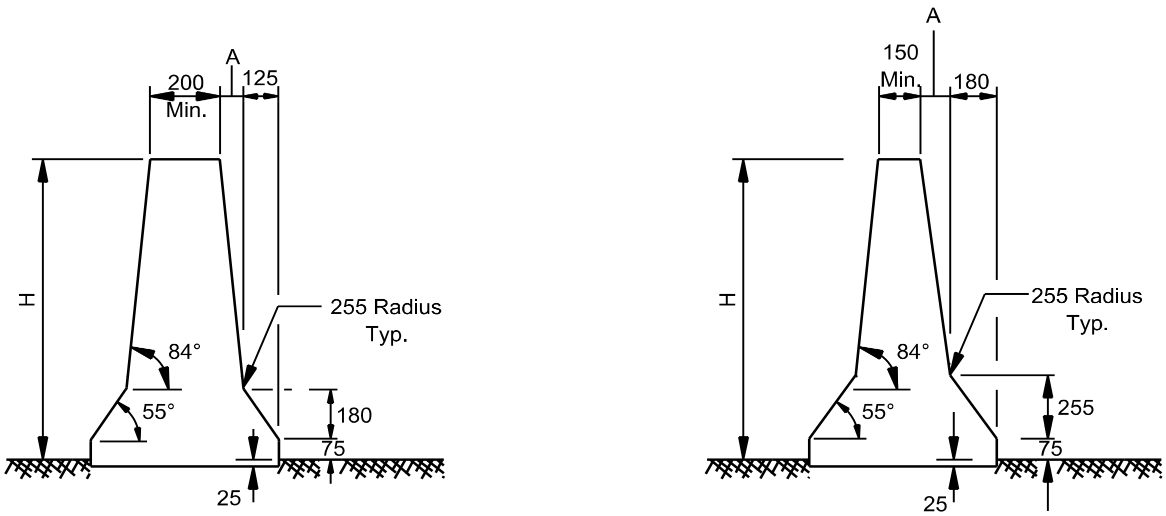
Traffic barriers include both longitudinal barriers and crash cushions. The primary function of longitudinal barriers is to redirect errant vehicles. The primary function of crash cushions is to decelerate errant vehicles to a stop. Longitudinal barriers are located along the roadside and in medians. Longitudinal barriers are generally denoted as one of three types: flexible, semi-rigid, or rigid. The major difference between these types is the amount of barrier deflection that takes place when the barrier is struck.

Flexible barrier systems undergo considerable dynamic deflection upon impact and generally impose lower impact forces on the vehicle than semi-rigid and rigid systems. The resistance is derived from the tensile force in the longitudinal member. For semi-ridge systems, the resistance is achieved through the combined flexure and tensile strength of the rail. A rigid system does not deflect substantially upon impact. During collisions, energy is dissipated by the raising and lowering of the vehicle and by deformation of the vehicle body. Concrete median barriers that are 810 mm high meet the TL-4 test levels, whereas, the 1070-mm high barriers are TL-5 barriers. For guidance on barrier test levels, see the Abu Dhabi Roadside Design Guide (28).

All barrier types have a zone of intrusion (ZoI) which ranges from around 0.5m for concrete barriers due to high sided vehicles leaning over during an impact, to around 1.6m for typical W-beam safety fence and over 2m for normal wire rope safety fence. Also refer TR-518.Figure 3-15 shows typical concrete barrier designs.

Important factors to consider in the selection of a longitudinal system include barrier performance, lateral deflection characteristics, and the space available to accommodate barrier deflection. In addition, the design should consider adaptability of system to operation transitions and end treatments and to the initial and future maintenance cost. Systems should ideally meet the requirements for MASH (the minimum acceptable standard is NCHRP-350) and it should be noted that systems worldwide are generally not tested at 140kph or 160kph speeds and therefore may not work as intended. Figure 3-16 illustrates a design for a cast-in-place curb to concrete barrier transition.

Figure 3-15: Concrete Median Barrier Details

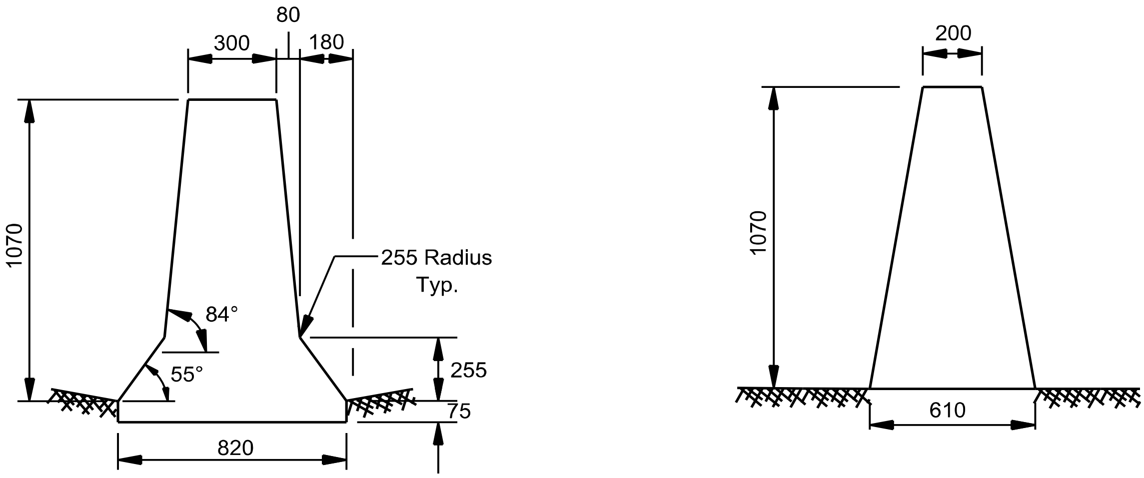


System	A	H
SGM10a	60	810
SGM10b	85	1070

Details of
F-Shape Median Barrier

System	A	H
SGM11a	50	810
SGM11b	80	1070

Safety Shape (NJ)
Median Barrier



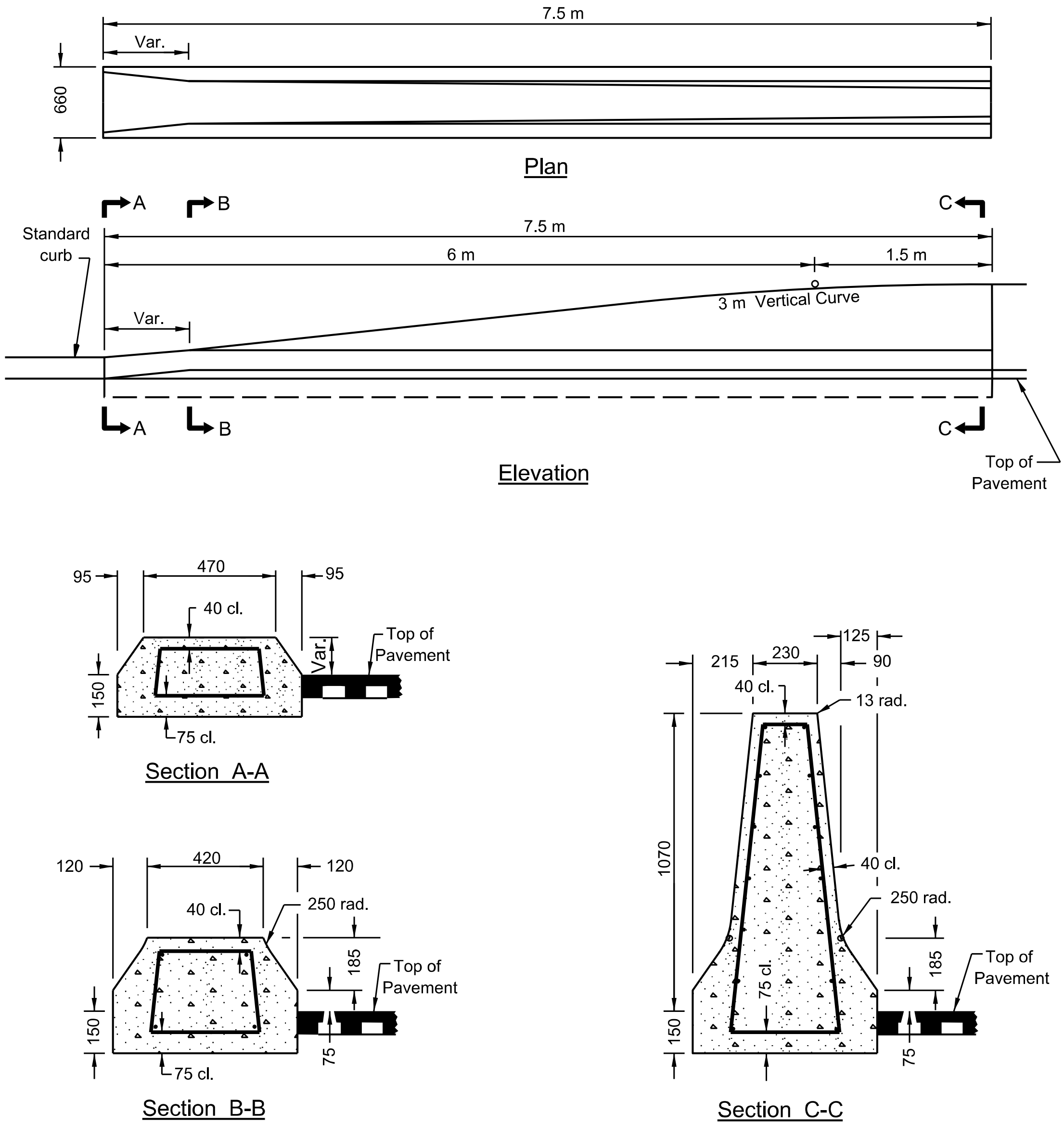
Tall Wall
Median Barrier

Texas Constant
Slope Barrier

SGM10a, SGM10b, SGM11a, and SGM11b are AASHTO designations for concrete median barriers. Option “a” is for an 810 mm high concrete median barrier and Option “b” is for a 1070 mm high barrier.

All dimensions are in millimetres unless otherwise noted.

Figure 3-16: Example Cast-in-Place Curb to Concrete Barrier Transition



All dimensions are in millimetres unless otherwise noted.

Roadway cross section designs significantly affect traffic barrier performances. Curbs, dikes, sloped shoulders, and stepped medians can cause errant vehicles to vault or submarine a barrier, or strike a barrier so the vehicle overturns. Optimum barrier system performance is provided by a relatively level surface in front of the barrier and, for semi-rigid and flexible barriers, beneath and behind the barrier. Where curbs and dikes are used to control drainage, they should be located flush with the face of the barrier or slightly behind it.

On the roadway approach to a bridge, the bridge railing is extended beyond the bridge with a roadside barrier. At the approach juncture between a bridge railing and the roadside barrier, an incompatibility may exist in the stiffness of the two barrier types. This stiffness should be gradually transitioned over a length to prevent the barrier system from pocketing or snagging an impacting vehicle. For barrier transition details, see the Abu Dhabi *Standard Drawings for Road Projects* (27).

3.6 Noise Control

3.6.1 General

Noise is defined as unwanted sound. Motor vehicles generate traffic noise from the motor, aerodynamics, exhaust, and interaction of the tires with the roadway. The designer should strive to minimise the radiation of noise into noise-sensitive areas along the roadway through location and design considerations. Noise attenuation may be inexpensive and practical if built into the design, and expensive if not considered until the end of the design process. An effective method of reducing traffic noise from adjacent areas is to design the highway so that some form of solid material blocks the light of sight between the noise source and the receptors (e.g. earth embankments, noise barriers). The designer should take advantage of the terrain in forming a natural barrier so that the roadway appearance remains aesthetically pleasing.

In terms of noise considerations, a depressed roadway section is the most desirable. Depressing the roadway below ground level has the same general effect as erecting barriers (i.e., a shadow zone is created where noise levels are reduced). Where a roadway is constructed on an embankment, the embankment beyond the shoulders will sometimes block the line of sight to receptors near the roadway, thus reducing the potential noise impacts. Where other road design measures are not appropriate, noise barriers may be considered.

3.6.2 Sound Barriers

The need to incorporate sound barriers in a road project is identified in a noise study, usually as part of the overall environmental assessment. A typical noise study report will indicate noise level criteria and any sound barriers required to achieve them, including their location, extent, and height. The design process usually involves consideration of options to ensure a suitable final design.

A sound barrier may take the form of an earth mound, a wall or solid fencing, or a combination of these.

3.6.2.1 Aesthetic Requirements

It is desirable that sound barriers are designed so that they:

- Are acceptable to the local community and local government authority, in terms of appearance, location, maximum height, and security issues for abutting property owners/occupiers.
- Are aesthetically pleasing and suit the landscape and local environment.
- Integrate planting or vegetated screening where space permits. Preference should be given to screening the full height of the wall, where suitable, to soften the appearance and to reduce graffiti and vandalism opportunities.
- Have materials and treatments that are contextually appropriate to the landscape setting.
- Use high-quality themed treatments to create visual interest where warranted (e.g. at highly visual locations and key focus points such as interchanges and entrances to a specific area).
- Retain views through the design and location of barriers where there are high-quality scenic views from the roadway.

3.6.2.2 Structural and Material Requirements

Sound barriers should be designed and constructed so that they:

- Are structurally sound.
- Are durable so that they perform satisfactorily with respect to degradation, corrosion, and appearance throughout their design life.
- Use acceptable materials such as brick, concrete, stone, and steel;
- Have appropriately designed and located footings to facilitate maintenance between the wall and the property boundary or be coincident with it.
- Have a logical termination point satisfying noise objectives as well as fitting with the road design.
- Have consistent colour.

The *Abu Dhabi Road Structures Manual* (29) discusses the structural design of sound barriers.

3.6.2.3 Other Requirements

Careful consideration should be exercised so that the construction and placement of noise barriers will not increase the severity of crashes that may occur. When designing the location of noise barriers, the designer should consider the following:

1. <u>Signs</u>. Make every effort to locate noise barriers to allow for sign placement to provide lateral offsets to obstructions outside the edged of the travelled way. It is recognised, however, that such a setback may sometimes be impractical. In these situations, provide the largest practical width commensurate with cost-effectiveness considerations.
2. <u>Sight Distances</u>. Horizontal clearances should be checked for adequate sight distances. Construction of a noise barrier should be avoided at a given location if it would limit stopping sight distance below minimum values. This situation could be particularly critical where the location of a noise barrier is along the inside of a curve.
3. <u>Roadside Barriers</u>. Some designs use a concrete safety shape as an integral part either of the noise barrier or as a separated roadside barrier. On non-tangent alignments, a separate concrete barrier may obstruct sight distance even though the noise barrier does not. In these situations, it may be appropriate to install a metal barrier rather than a concrete barrier.
4. <u>Gore Noses</u>. Exercise care when locating noise barriers near gore areas. Barriers at these locations should begin or terminate at least 60 m from the theoretical nose.
5. <u>Maintenance</u>. Sound barriers should be designed to require a low level of maintenance. The barrier should be graffiti resistant so that graffiti can be readily removed. Ensure adequate access for maintenance workers and machinery is available, preferably to both sides of the barrier, but at least to one side

3.7 Sand Control

3.7.1 General

Blowing sand present a unique challenge to designers. Blowing and shifting sand, prevalent in many areas, not only affects maintenance costs, but also driver safety. It is virtually impossible to eliminate all the problems associated with blowing sand. The designer can minimise these problems by understanding how sand moves and applying techniques that have successfully reduced the problems associated with windblown sand. Figure 3-17 illustrates sand covered roadways.

Many of the principles described in this section are based upon assumptions of uniform wind direction, level surfaces, etc. However, each field location and design requires a special study with regard to prevailing wind directions, wind velocities, topography, natural obstructions, etc. The designer, geotechnical engineer, and construction supervisor must to be aware of the various principles involved in sand movement and apply these design techniques as necessary in the various field applications.

Figure 3-17: Sand Covered Roadways

3.7.2 Sand Movement and Deposition

In the absence of moisture or other cohesive binders, the movement of sand particles is largely a function of wind velocity and sand particle size. Numerous factors and variables contribute to sand movement. Formulas have been developed to predict certain movements, to estimate quantities being moved or to estimate rate of advancement of dune formations. However, these formulas are not provided here. All of these formulas are founded on certain assumptions for usual ideal situations and can only approximate the actual behaviour of sand in nature.

Sand movement can be described in the following ways:

1. <u>Suspension</u>. The movement of sand particles in suspension occurs when the upward wind velocity components exceed the downward gravitational forces of the sand grain size. The particles remain in suspension and assume the velocity of the wind. This mode of transport manifests during dust storms. It is generally restricted to particles smaller than 0.08 mm.

2. Saltation. Saltation is the term used to describe the bounding motion of medium-sized sand grains during a moderate wind. At what is known as the fluid threshold, wind velocity is capable of lifting a sand grain and carrying it forward in the direction of the wind. When the particle falls back to the surface due to gravity, upon impact the grains either bounce up again, or hit and eject other grains from the surface that, in turn, move forward bounding and ejecting still other grains. Once the process has begun, it will propagate downwind and intensify as the wind velocity increases.

The majority of sand movement (approximately 90%) occurring in saltation takes place within 30 mm of a loose sand surface. Salt in saltation accounts for the greatest amount of sand movement in dune areas. In naturally graded dune sand with an average grain size of approximately 0.25 mm, saltation accounts for approximately 75% to 80% of sand movement.

3. Creep. Creep is the movement of sand particles that are otherwise too large to be lifted or rebounded from the surface. It is the slow forward movement of larger size grains that are pushed forward by the wind or by impacting sand grains, but do not become airborne themselves.

4. Gravitational Sliding. Gravitational sliding is a form of creep occurring primarily in dune areas. It characterises the gradual sloughing off the face of a dune when the accumulation at the crest of the dune exceeds the angle of repose of the sand. Gravitational forces cause forward and downward movement of the dune face. Similar movement occurs where erosion undermines sand formation.

3.7.3 Location

If possible, avoid locating roadways in sand dune areas. Where this is not reasonably, conduct an economic comparison to evaluate a more circuitous route that would bypass to the windward direction of the prevailing wind or dune area. This analysis should include not only construction costs, but also projected maintenance costs over the design life of the project.

1. Roadway Location. Where it is necessary to traverse a sand dune area, several basic principles should be observed:

- Locate the roadway alignment to the windward side of large dunes or isolated dune fields.
- Alignment paralleling the prevailing wind direction presents less potential for sand accumulation.
- Keep profile grade lines above the elevation of upwind topography as practical.
- Avoid through cuts and cuts to the upwind slope as practical.
- Design horizontal curves so that the superelevation does not exceed 5%.

2. Interchanges and Intersections. Interchange or intersection locations in sand dune areas are not desirable. When necessary, the principles discussed in Chapter 10 “Intersections” and Chapter 12 “Interchanges” are equally applicable. In addition,

special evaluation and treatment may be necessary. In interchange areas particularly, there is no way to avoid the grade differential between roadways. Aligning the bridge opening so that the prevailing wind can pass through it will help, but this may not be feasible. In these instances, the best long-range solution may be to install several series of windbreaks sufficiently upwind to unladen the wind of its sand before entering the interchange area. These windbreaks may consist of sand fences or rows of tamarisk trees.

3.7.4 Design Elements

Sand will be deposited whenever wind velocities fall below that necessary to keep the sand particles moving. Anything that disrupts the normal laminar flow of the wind can create conditions inductive to sand deposition. For this reason, it is extremely important to maintain a uniform aerodynamic profile across the roadway cross section.

The paved surface of the roadway presents less friction than the desert floor and wind speeds are increased across the paved area. Elevating the roadway profile 1.5 m to 2.0 m above the surrounding terrain together with the increased wind velocity across the paved surface helps keep the roadway surface clear of sand. In order not to create turbulences, it is necessary to maintain gentle side slopes.

3.7.4.1 Cross Section

Some of the more important considerations in cross section design elements in areas of blowing sand are as follows:

- Cut slopes should be 1V:10H or flatter, regardless of type of material excavated.
- Fill slopes should be 1V:6H or flatter when the maximum height of fill does not exceed 4 m.
- The maximum superelevation rate should be 5%.
- Use flat grates on drop inlets.
- On freeways and expressways, provide wide shallow medians to eliminate the need for median barriers.
- To assist in keeping a median clear of sand, consider paving the median.
- Where possible, do not use curbs, steel-beam guardrails, or concrete barriers.
- Wire Rope Safety fence is the preferred RRS in areas subject to blowing sand.

Where the elevation of an upwind dune crest is above the elevation of the roadway, whether the roadway is in cut or fill, it is necessary to determine whether the distance to the crest from edge of roadway is greater or less than 20 times the elevation difference in order to determine the method of stabilization. See Figure 3-18. Bridge openings at grade-separation should be as open as possible. The elimination of bridge piers is desirable. Bridge railing should be the open tubular type as opposed to concrete barrier.

Figure 3-18: Sand Dunes

3.7.4.2 Roadside Appurtenances

Every roadway involves more than just the roadway template itself. Signs, signals, fences, guardrails, etc., become a necessary part of the final facility and provide various degrees of guidance and safety to the travelling public. Wind turbulence is caused wherever there is an obstruction to the normal laminar wind flow. The turbulence created results in some areas of low velocity that can result in sand deposition, as well as other areas of high velocity, which may cause erosion.

In general, minimise the installation of roadside appurtenances. Where feasible, mount signs on a grade separation bridge rather than creating an additional supporting structure. Large supporting structures will require stabilization of an area at least 1 m in all directions from the support to minimise erosion. In the case of small supports and fence posts, the top of concrete foundations should be more than 100 mm below the sand surface to minimise the effects of cavitation.

A special strained wire fence has been developed for use as right-of-way fence in the sand areas. This fence is more open than chain link and other types of fence that have a tendency to trap paper and debris, which can cause sand depositions. In areas of active dune movement, provide a stabilized strip of at least 1 m along fence lines. Wide shallow medians should be used in sand areas. Where a median barrier is required in the sand areas, a modified strained wire fence has been developed that incorporates two 19-mm diameter cables as a deterrent to crossing or cutting. A minimum median width of 8 m is required to use this fence.

3.7.5 Dune Stabilization

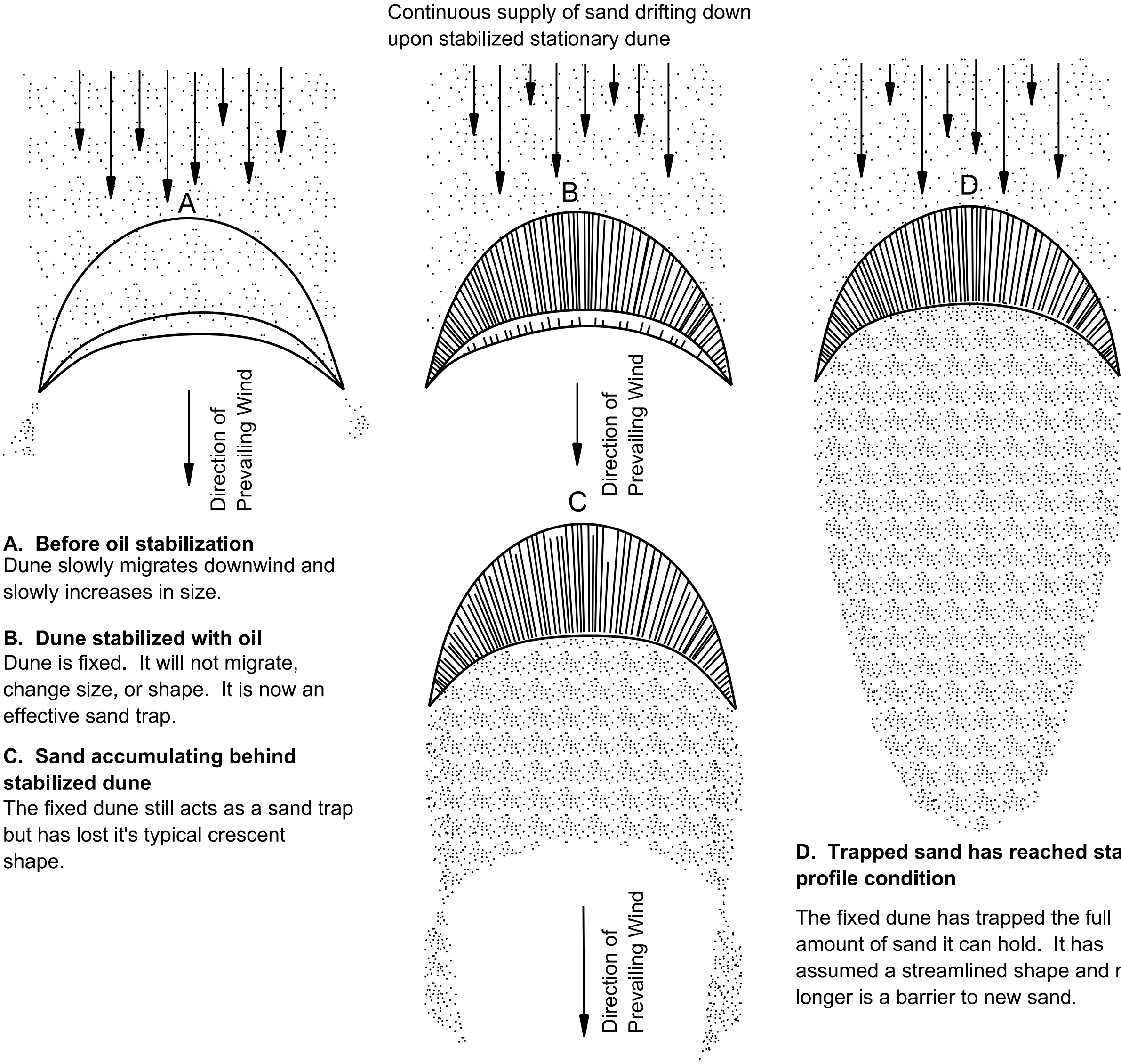
Stabilizing dunes and other loose sand areas involves the process of immobilizing the sand. In the case of a barchans dune, stabilizing the crest and wings of the dune actually causes the trough of the dune to trap sand. Sand will accumulate in this area until an aerodynamic profile is established. Once established, the profile will remain essentially stable because the stabilized crest cannot move forward by saltation or creep. The wind will maintain the uniform profile even without the subsequent stabilization of the trapped sand. Figure 3-19 shows how oil stabilized crescent-shaped dune acts to trap sand.

Dune stabilization may be performed in a variety of ways. The Abu Dhabi *Standard Specifications* include those methods currently considered acceptable. These methods include the application of gravel blankets that provide a protective layer that is not as susceptible to erosion or saltation. The cohesion of the dune sands can be increased by the application of crude oil, crude oil blends, and emulsified asphalts. Certain chemical stabilizers can also be used to increase cohesion; however, their effect is usually only temporary. Vegetation can also be used to stabilize dune areas; however, the arid environment makes use of this method extremely limited. Sand fences are also effective, but require continual maintenance.

Stabilizing large areas with complete coverage can be quite expensive. Where large areas are involved, strip stabilization has been found successful in reducing costs while maintaining the stability of the dune feature. When using strip stabilization to immobilise dune features, ensure the strips are perpendicular to the direction of the prevailing winds. The width of untreated area between the stabilized strips should not exceed 4 m in order to maintain the areas effectiveness. Wider untreated areas will encourage sand movement and erosion of the treated strips. Strip stabilization may be performed by spray application or by injection methods. Because injection methods provide for greater control with respect to the applied width and depth of penetration, the patterns give favour to this method and less stabilizing materials are used. This savings in stabilizing materials is offset by additional equipment costs.

When spray stabilization is required, specify the rate of application of the material to be used, but do not specify a minimum depth of penetration. The depth of penetration of spray materials varies with the type of material being sprayed, the type and natural density of the dune sand, and to a lesser extent on the rate of application. Density tests taken on Barchan dunes have ranged from 105% at the crest to 85% in the advancing horns.

Figure 3-19: Stabilizing Dunes



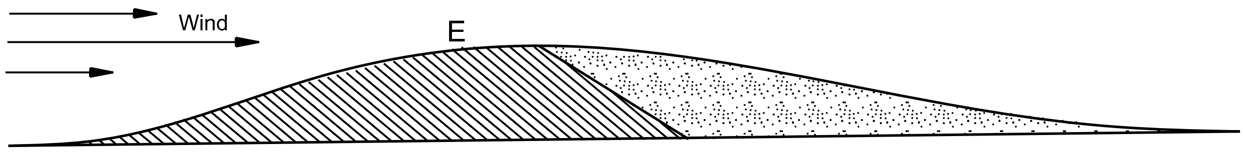
A. Before oil stabilization
Dune slowly migrates downwind and slowly increases in size.

B. Dune stabilized with oil
Dune is fixed. It will not migrate, change size, or shape. It is now an effective sand trap.

C. Sand accumulating behind stabilized dune
The fixed dune still acts as a sand trap but has lost it's typical crescent shape.

D. Trapped sand has reached stable profile condition

The fixed dune has trapped the full amount of sand it can hold. It has assumed a streamlined shape and no longer is a barrier to new sand.



E. Side view of ultimate stable condition

Trapped sand is impounded; however, the pile is no longer capable of stopping new sand drifting downwind.

Source: (30)

The use of sand fence is effective in immobilizing dunes, but requires frequent maintenance. As each fence installation is buried, it must be extended in height. This raising in height can effectively result in the trapping of many thousands of cubic meters of sand, if properly maintained. Consider providing a sand fence on the upwind side of an interchange in a dune area.

For these installations, provide two or more parallel rows located perpendicular to the prevailing wind. The upwind fence will silt in very fast and will need to be raised first. The second and third fence downwind can be used to measure (gage) the rate of accumulation in order to plan future fence installations.

3.7.6 Dune Destruction

In certain instances, it may be desirable to destroy a dune formation. The forces of the wind can be used to erode and lower a dune formation by orienting strip stabilization in a direction parallel to the prevailing wind. One application where dune destruction might be beneficial is where an overhead power line is between the roadway and an approaching dune. Destruction or lowering of this dune would preserve the clearance of the power line. For smaller dune features, differential oiling of either the wings or the centre section of the crescent-shaped dunes can also be effective.

Exercise caution when dune destruction is used, because the eroding sand must pass across the roadway facility. Aerodynamic slopes and lack of obstruction is necessary so that the moving sand is facilitated in crossing the roadway and not encouraged to be deposited upon the roadway. Also, evaluate the downwind side of the roadway so that the increased sand coming to this area does not cause other problems.

3.8 Right-of-Way

The minimum right-of-way width for all functional classifications will be the sum of the travel lanes, outside shoulders, median width (if applicable), service areas, plus the necessary width for fill and cut slopes or for roadside clear zones, whichever is greater. Desirably, the right-of-way width will accommodate the anticipated ultimate development of the roadway facility.

The right-of-way width should be uniform, where practical. In urban areas, variable widths may be necessary due to existing development; varying side slopes and embankment heights may make it desirable to vary the right-of-way width; and right-of-way limits will likely have to be adjusted at intersections and freeway interchanges. The designer should also consider the following special right-of-way controls:

1. <u>Intersections and Horizontal Curves</u>. At horizontal curves and intersections, additional right-of-way may be warranted to ensure that the necessary sight distances are available in the future.

2. <u>No Available Right-of-Way</u>. In areas where the necessary right-of-way widths cannot be reasonably obtained, the designer may consider using steeper slopes, revising grades, or using retaining structures.

3. <u>Services Area</u>. Although the cross section guidelines provide for service corridors (see the Abu Dhabi *Right-of-Way Utilities Distribution Manual* (31)), the utility authorities may seek wider widths than is available. Under these circumstances, it is important to reach an agreement with all the relevant parties early in the design process before the design is set.

3.9 Utilities

3.9.1 Types

In general, all utilities (both public and private) may be classified as pipelines, electric power lines, or communication lines:

1. Pipelines. Pipelines may be further divided according to the products they transmit:

   - gas lines (i.e. high-pressure or medium-pressure transmission lines, distribution lines, vacuum lines or gathering lines);
   - oil transmission and distribution lines;
   - oil transmission and distribution lines (i.e. gathering lines, waste lines or salt water lines);
   - water transmission and distribution lines;
   - sanitary sewer lines;
   - storm water (gravity and pressure) lines;
   - irrigation lines; and
   - other miscellaneous pipelines.

2. Electric Power Lines. Electric power lines may be transmission lines, distribution lines, or service lines. These lines may be aerial, underground or a combination of both.

3. Communication Lines and/or Facilities. Communications lines and/or facilities may be aerial, underground, a combination of both or electronic transmission towers. Their function may encompass local or regional service.

3.9.2 Location

The road designer is generally responsible for initially determining utility locations. The following are general rules of good and poor practices for determining utility locations:

1. Future Roadway Improvements. Locate utility lines to minimise the need for later adjustment to accommodate future roadway improvements and to permit servicing these lines with minimum interference to roadway traffic.

2. Uniform Alignment. Locate longitudinal installations on uniform alignment as near as practical to the right-of-way line to provide a safe environment for traffic operations and preserve space for future roadway improvements or other utility installations.

3. Roadway Crossings. To the extent feasible, utilities crossing the roadway should cross on a line generally perpendicular to the roadway alignment.

4. Clear Zones. Ensure the horizontal and vertical location of utility lines within the roadway right-of-way limits conform to the clear zone criteria. The location of aboveground utility facilities should be consistent with the clearances applicable to other roadside obstacles.

5. Undesirable Underground Crossings. Avoid conditions that are generally unsuitable or undesirable for underground utility line crossings. These include, but are not limited to, locations:

- in deep cuts;
- near footings of bridges and retaining walls;
- at cross drains where flow of water, drift, or stream bed load may be obstructed;
- within basins of an underpass drained by a pump if the pipeline transports a liquid or liquefied gas; and
- in wet or rocky terrain where it will be difficult to attain minimum cover.

6. Roadway Structural Integrity. On longitudinal installations, utility locations parallel to the roadway at or adjacent to the right-of-way line are preferable to minimise interference with the safe operation of the roadway, and roadway drainage and the structural integrity of the travelled way, shoulder, and embankment.

7. Clearances. Vertical and horizontal clearances between a pipeline and a structure or other roadway or utility facilities should be sufficient to permit maintenance of the pipeline and other facilities.

3.9.3 Design Considerations

The road designer should ensure that the proposed road design is consistent with the practicalities of utility work. Therefore, the designer should consider the following:

1. Clearance and Cover. The road designer should consult with the utility authorities to ensure proper clearance and cover is met.

2. Aerial. The designer should consider all overhead utilities and should consult with the power company to ensure aerial electric power lines and/or communication lines are not jeopardized by construction.

3. Detours. Awareness of potential utility conflicts is extremely important when constructing detours near the right-of-way line or on temporary right-of-way.

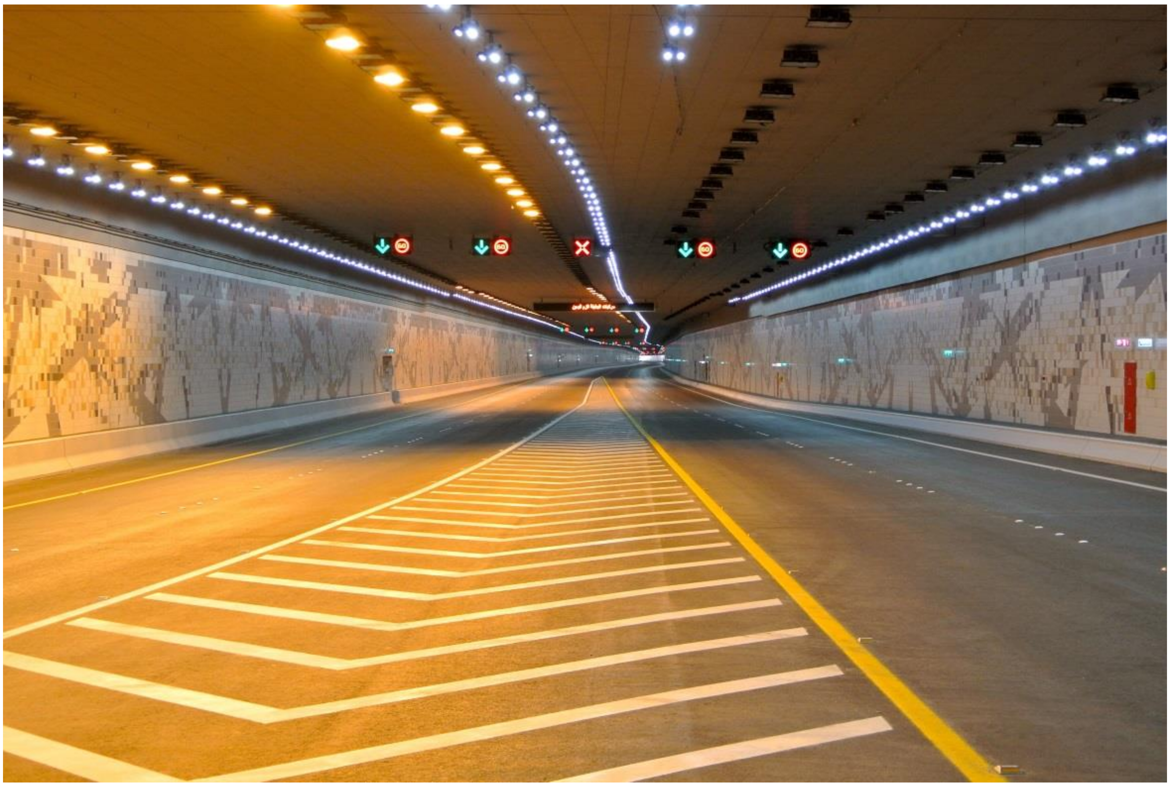
Additional guidance on accommodating utilities within the right-of-way is provided in the Abu Dhabi *Right-of-Way Utilities Distribution Manual* (31), DMT *Utility Corridors Design Manual* (32), AASHTO *A Policy on the Accommodation of Utilities within Freeway Right-of-Way* (33), and AASHTO *A Guide for Accommodating Utilities within Highway Right-of-Way* (34).

3.10 Tunnels

In some situations, it may be determined to be advantageous to place the roadway in a tunnel to go under or through a natural obstacle or to minimise the impact of road or street. See Figure 3-20 for photos of tunnels in Abu Dhabi. General considerations under which tunnel construction may be warranted include:

- long, narrow terrain ridges where a cut section may either be costly or carry environmental consequences;

Figure 3-20: Tunnels



- narrow rights-of-way where all of the surface area is needed for street purposes;
- large intersection areas or a series of adjoining intersections on an irregular or diagonal street pattern;
- near railroad yards, airport runways, or similar facilities;
- parks or similar land uses, existing or planned; or
- locations where right-of-way acquisition costs exceed cost of tunnel construction and operation.

General construction and design features of tunnel sections are discussed in the following sections. It is not intended that these sections be comprehensive on the subject of the design of roadway runnels. The design of tunnels require specialised engineering experience in soils, structures, lighting, ventilation, pumping, mechanical, and electrical, which are beyond the scope of this *Manual*.

3.10.1 Types of Tunnels

Tunnels can be classified into two major categories: (1) tunnels constructed by mining methods, and (2) tunnels constructed by cut-and-cover methods.

The first category refers to those tunnels that are constructed without removing the overlying rock or soil. Usually this category is subdivided into two very broad groups according to the appropriate construction method. The two groups are named to reflect the overall character of the material to be excavated — hard rock and soft ground.

Of particular interest to the designer are the structural requirements of these construction methods and their relative costs. Generally, hard rock tunnelling is less expensive than soft-ground tunnelling. A tunnel constructed through solid, intact, and homogenous rock will normally represent the lower end of the scale with respect to structural demands and construction costs. A tunnel located below water in material that needs immediate and heavy support will involve extremely expensive soft-ground tunnelling techniques (e.g., shield and compressed air methods).

The second category of tunnel classification deals with the two types of tunnels that are constructed from the surface: trench and cut-and-cover tunnels. The latter are used exclusively for subaqueous work. In the trench method, prefabricated tunnel sections are constructed in shipyards or dry docks, floated to the site, sunk into a dredged trench, and joined together underwater. The trench is then backfilled.

The cut-and-cover method is by far the most common type of tunnel construction for shallow tunnels, which often occurs in urban areas. This method consists of excavating an open cut, building the tunnel within the cut, and backfilling over the completed structure; see Figure 3-21.

Figure 3-21: Cut and Cover Tunnels

3.10.2 Design Considerations

Tunnels should be as short as practical because the feeling of confinement and magnification of traffic noise can be unpleasant to motorists, and the high construction costs of tunnels. The designer should consider the following guidelines:

1. Horizontal Alignment. Keep the tunnel on a tangent alignment as much as practical to minimise the tunnel length and improve operating efficiency. Tunnels designed with extreme curvature may result in limited stopping sight distance. Ensure horizontal sight distance is available on the inside of horizontal curves. This may require widening the shoulder on the inside of the curve to obtain the necessary sight distance.

2. Vertical Alignment. Grades in tunnels should be determined primarily based on driver comfort while striving to reach a point of economic balance between construction costs and operating and maintenance expenses. Many factors have to be considered in tunnel lengths and grades and their effects on tunnel lighting and ventilation. For example, lighting expenses are highest near portals and depend heavily on availability of natural light and the need to make a good light transition. Ventilation costs depend on length, grades, natural and vehicle-induced ventilation, and air-quality constraints.

3. Vertical Clearance. Provide a minimum vertical clearance of 6.5 m. The 6.5-m clearance is also required to the bottom of overhead signs and ventilation systems. For urban areas, the minimum vertical clearance is 6 m.

4. Signing. Normal vertical and lateral clearances in tunnels are usually insufficient for signing. Therefore, design the overall roadway to avoid the need for guide signs within tunnels. Ensure exit ramps are located a sufficient distance downstream (minimum of 300 m) from the tunnel portal so that needed guide signs may be placed between the tunnel and the point of exit.

5. Weaving. It is highly undesirable that traffic be expected to merge, diverge, or weave within a tunnel (e.g. between two closely spaced interchanges). Therefore, avoid providing exit or entrance ramps within tunnels, where practical.

3.10.3 Tunnel Cross Sections

Full left- and right-shoulder widths of the approach freeway desirably should be carried through the tunnel. The need for added lateral space is greater in tunnels than under separation structures because of the greater likelihood of vehicles becoming disabled in the longer lengths. If shoulders are not provided, intolerable delays may result when vehicles become disabled during periods of heavy traffic. However, the cost of providing shoulders in tunnels may be prohibitive. The determination of the width of shoulders to be provided in a tunnel should be based on an in-depth analysis of all factors involved. Where it is not practical to provide shoulders in a tunnel, arrangements should be made for around-the-clock emergency service vehicles that can promptly remove any stalled vehicles.

Figure 3-22 illustrates the typical cross sections for three-lane tunnels. The minimum roadway width between curbs, as shown in Figure 3-22(a), should be at least 0.6 m greater than the approach travelled way. The curb or sidewalk on either side should be a minimum of 0.6 m. The total clearance between walls of a three-lane tunnel should be a minimum of 12.75 m (assuming three 3.65-m wide travel lanes). The roadway width and curb or sidewalk width can be varied as needed within the 12.75 m minimum wall clearance; however, each width should not be less than the minimum value stated above.

Figure 3-22(b) illustrates the desirable section with three 3.65-m travel lanes, 3.0-m right and left shoulders, and a 0.9-m curb or sidewalk on each side. The roadway width may be distributed to either side in a different manner if needed to better fit the dimensions of the tunnel approaches.

Normally, pedestrians are not permitted in freeway tunnels; however, space should be provided for emergency walking and for access by maintenance personnel. Raised sidewalks, 0.9 m wide, are desirable beyond the shoulder areas to serve the dual purpose of a safety walk and a buffer to prevent the overhang of vehicles from damaging the wall finish or the tunnel lighting fixtures. Separate tunnels may be warranted for pedestrians or other special uses (e.g. cyclists routes).

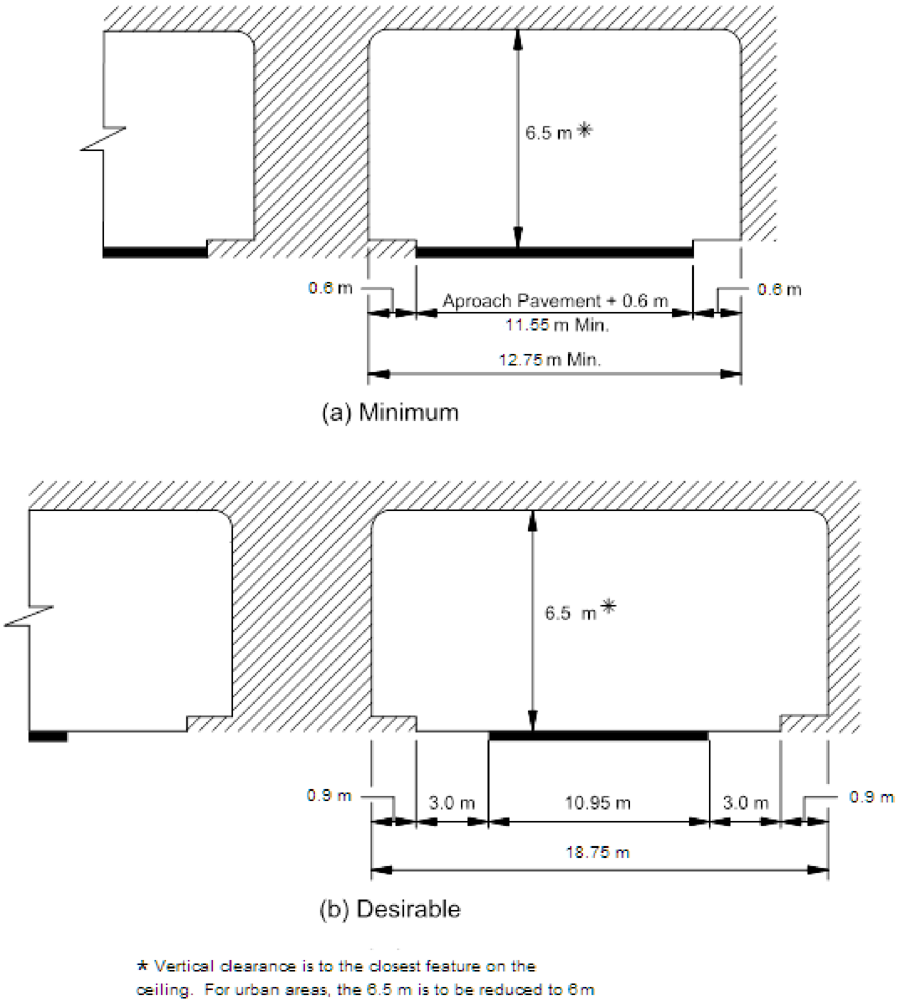
3.11 Wildlife Crossings

Wildlife crossings are structural passages beneath or above roadways that are designed to facilitate safe wildlife movement across roadways. Wildlife crossings coupled with roadside fencing are ways to increase road permeability and habitat connectivity while decreasing wildlife-vehicle collisions. Wildlife crossing is the umbrella term encompassing underpasses, overpasses, ecoducts, green bridges, amphibian/small mammal tunnels, and wildlife viaducts. All of these structures are designed to provide semi-natural corridors above and below roads so that animals can safely cross without endangering themselves and motorists.

The underpass should be designed to accommodate the species at risk of habitat fragmentation. If practical, the underpass may also accommodate a wider range of wildlife. There are a range of features within and adjacent to underpasses that can enhance their performance for some species. Factors include:

- Using wildlife fencing to guide animals to the underpass,
- Providing habitat/cover appropriate to the target species leading to a culvert inlet and/or outlet,

Figure 3-22: Tunnel Cross Section



- Using culverts large enough and short enough to allow daylight to be viewed through the structure, and

- Using the natural ground surface or providing natural materials within the culvert/ underpass.

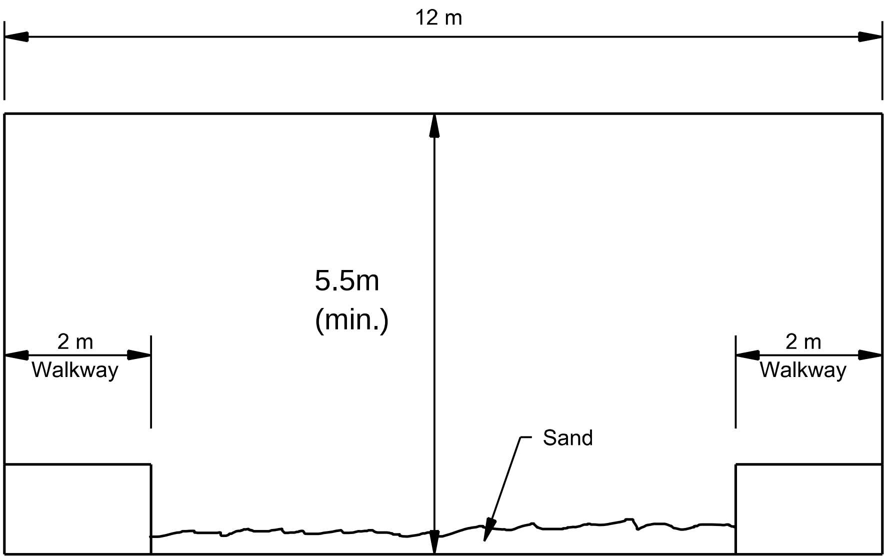
Whenever wildlife underpasses are part of a project, advice should be obtained on the needs of individual species considering:

- habitat requirements,
- migratory patterns of the species,
- underpass size and spacing requirements, and
- underpass slope/grade requirements for the species.

Figure 3-23 shows a wildlife underpass that will also serve as a waterway passage during wet periods. Figure 3-24 shows a cross section of a typical camel crossing.

Figure 3-23: Wildlife Underpass

Figure 3-24: Camel Underpass



3.12 Cul-De-Sacs and Turnarounds

A local street, open at one end only, should have a special turning area at the closed end. This turning area may be an “L,” “T,” or circular shape cul-de-sac with dimensions as appropriate for the type of vehicle expected. The commonly used circular form should have a minimum outside radius of 10 m in residential areas and 15 m in commercial and industrial areas.

A dead-end street narrower than 12 m should be widened to enable passenger vehicles, delivery trucks, and emergency vehicles to make U-turns or at least turn around by backing only once. Typically, the design is circular pavement, symmetrical about the centreline of the street sometimes with a central island, as shown in Figure 3-25(a), which also shows minimum dimensions for the design vehicles. Although this type of cul-de-sac operates satisfactorily, improved operation is obtained if the design is offset so that the entrance-half of the pavement is in line with the approach-half of the street, as shown in Figure 3-25(d). One steering reversal is avoided on this design. Where a radius of less than 15 m is used, provide sloping curbs on the island to permit manoeuvring of an occasional oversized vehicle.

An all-paved plan, as opposed to an island configuration, with a 10 m outer radius, shown in Figure 3-25(e), requires little additional paving. If the approach pavement is at least 10 m wide, the result is a cul-de-sac where passenger vehicles can make the customary U-turn and SU-9 design trucks can turn by backing only once. A radius of about 12 m enables a WB-15 vehicle to turn around by manoeuvring back and forth.

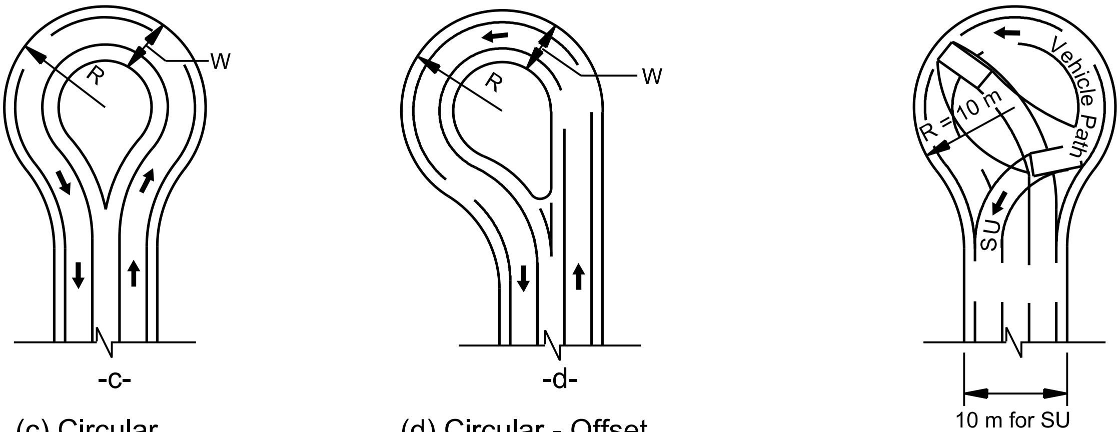
Other variations or shapes of cul-de-sacs that include right-of-way and site controls may be provided to permit vehicles to turn around by backing only once. Several types (e.g., Figure 3-25(f), Figure 3-25(g), and Figure 3-25(i)) may also be suitable for alleys. The geometry of a cul-de-sac should be altered if adjoining residences also use the area for parking. Alleys without a connection to a street or another alley should include a turning area at the end of the alley as shown in Figure 3-26. Figure 3-26 also may be suitable for application on some very low-volume roads as well.

Generally, streets with cul-de-sacs should not be greater than 300 m in length. Cul-De-Sacs and Turnarounds should be designed in a way that discourages vehicles from mounting the kerb and using the Non-Motorised users (NMU) path to access nearby through roads;

Figure 3-25: Cul-de-sacs

(a) (b)

Design Vehicle	W	L
P	10 m	20 m
SU	15 m	30 m

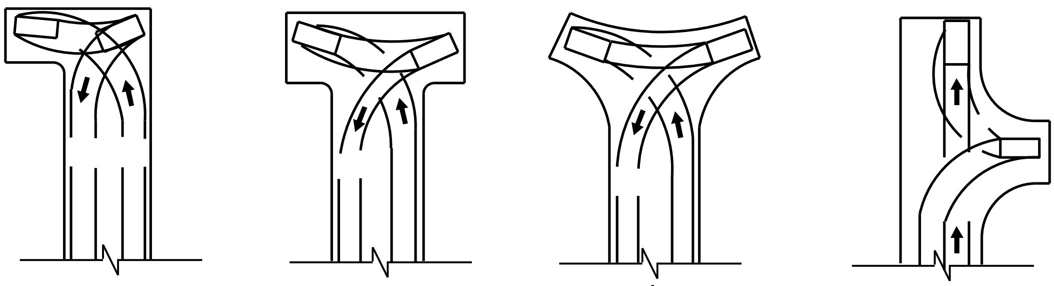


(c) Circular

(d) Circular - Offset

(e) Circular - All Paved

Design Vehicle	R	W
P	10 m	6 m
WB-12	13 m	8 m
SU & WB-15	15 m	10 m



-f- -g- -h- -i-

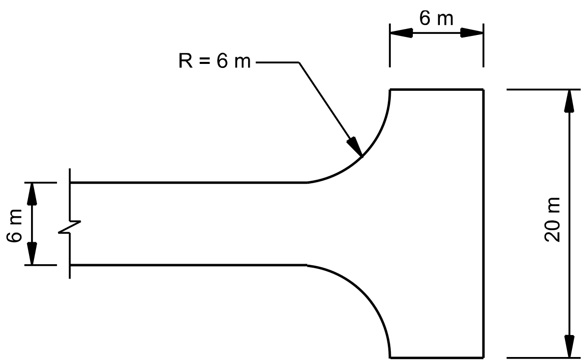
(f) L - Type

(g) T - Type

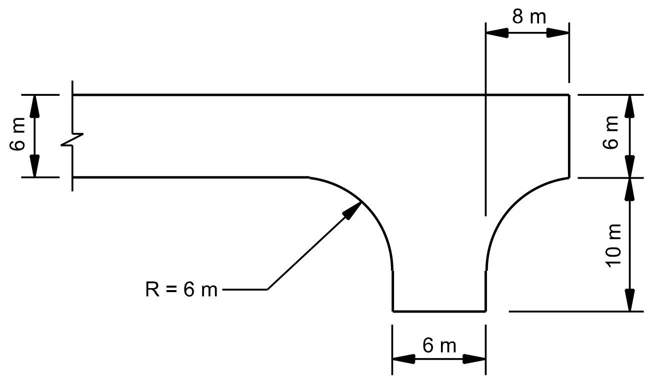
(h) Y - Type

(i) Branch

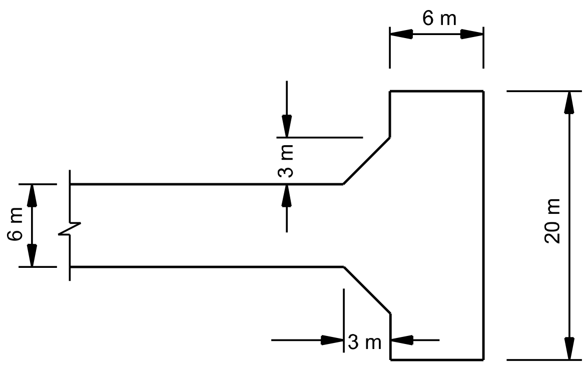
Figure 3-26: Turnarounds



Standard Turning Area



Turning Area



Standard Cut Corners

4 SIGHT DISTANCE

4.1 Overview

Sight distance is defined as the distance available on a length of road ahead that is visible to the driver. Road users must be able to see far enough forwards to carry out manoeuvres safety. The following manoeuvres are particularly relevant:

- stopping prior to reaching a stationary obstruction,
- overtaking another vehicle on an undivided road,
- making a decision where a choice of actions presents itself, and
- turning at or crossing intersections.

These are reflected in the criteria:

- Stopping Sight Distance (SSD),
- Decision Sight Distance (DSD),
- Passing Sight Distance (PSD), and
- Intersection Sight Distance (ISD).

Sight distances are measured from the driver’s eye in a straight line to an object on the roadway.

On horizontal curves, the lane nearest to the centre of the curve is normally the most critical. Consideration of sight distance may lead to a requirement for roadside objects on the inside of a curve to be set back from the edge of the travelled way by a greater amount than would be normal on a straight alignment. Chapter 5 “Horizontal Alignment” contains guidance on applying sight distance criteria on horizontal curves.

4.2 Eye and Object Heights

A critical element in determining the available sight distance is the viewpoint from the driver’s eye, which varies in height depending on the type of vehicle and the height and posture of the driver. For design purpose, a passenger car eye height is 1080 mm. The vast majority of passenger car drivers’ eye heights exceed this value, and it is likely that for lower values, the vehicles concerned tend to be higher-performance cars with superior braking and acceleration performance. If there are a sufficient number of trucks to warrant their consideration, use an eye height of 1800 mm for buses and single unit trucks, or 2330 mm for tractor/semitrailers.

For SSD and DSD, the object is considered the taillights of the ahead vehicle. A height of 600 mm from the pavement surface is used for the object height. For PSD and ISD, the object height is considered another passenger car and is assumed to be 1080 mm. An object height of 1080 mm assumes that a sufficient portion 225 mm of an oncoming passenger car must be seen to identify it as an object of concern by the driver. Using the 1080 mm height for both vehicles assumes that each driver can see and recognise the other vehicle.

4.3 Stopping Sight Distance

Stopping sight distance (SSD) is the sum of the distance travelled during a driver's perception/reaction, or brake reaction, time and the distance travelled while decelerating to a stop. Equation 4.1 calculates SSD for passenger cars.

Equation 4.1: SSD Level Grade

where:	SSD	=	stopping sight distance, m
	V	=	design speed, km/h
	t	=	brake reaction time, 2.5 s
	a	=	driver deceleration, m/s^2

The following briefly discusses the theoretical rationale for each assumption within the SSD model for passenger cars:

1. Brake Reaction Time. This is the time interval between when the obstacle in the road can first be physically seen and when the driver first applies the brakes. Based on several studies of observed driver reactions, the assumed value is 2.5 seconds. This time exceeds the necessary reaction time for nearly all drivers, including the older driver.

2. Braking Action. The braking action is based on the driver's ability to decelerate the vehicle while staying within the travel lane and maintaining steering control during the braking manoeuvre. A deceleration rate of 3.4 m/s^2 is considered comfortable for 90% of passenger cars drivers.

3. Design Speed. Use the roadway design speed as the driver's initial speed.

4.3.1 Passenger Cars (Level Grade)

Table 4-1 provides stopping sight distances for passenger cars on grades less than 3%.

4.3.2 Grade-Adjusted SSD

The longitudinal gradient of the roadway impacts the distance needed for vehicles to brake to a stop. Equation 4.1 is modified as shown in Equation 4.2 to calculate the adjusted SSD for grades equal to or greater than 3%.

Table 4-1: Stopping Sight Distance (Passenger Cars – Level Grade)

Design Speed (km/h)	Brake Reaction Distance[1] (m)	Braking Distance[2] (m)	Stopping Sight Distance[3]	
			Calculated (m)	Design (m)
20	13.9	4.6	18.5	20
30	20.9	10.3	31.2	35
40	27.8	18.4	46.2	50
50	34.8	28.7	63.5	65
60	41.7	41.3	83.0	85
70	48.7	56.2	104.9	105
80	55.6	73.4	129.0	130
90	62.6	92.9	155.5	160
100	69.5	114.7	184.2	185
110	76.5	138.8	215.3	220
120	83.4	165.2	248.6	250
130	90.4	193.8	284.2	285
140[4]	97.3	224.8	322.1	325

Notes:

1. *Brake reaction distance is based on 2.5 s.*
2. *Driver deceleration is based on a rate of 3.4 m/s^2.*
3. *These SSD are for grades < 3%.*
4. *Estimated by DMT.*

Source: (1)

$$\mathbf{SSD = 0.278Vt + \frac{V^2}{254\left(\frac{a}{9.81} \pm G\right)}}$$

Equation 4.2: SSD Graded Adjusted

where: SSD = stopping sight distance, m
V = design speed, km/h
t = brake reaction time, 2.5 s
a = driver deceleration, m/s^2
G = grade expressed as a decimal (+ for upgrades and – for downgrades).

Table 4-2 presents the grade adjusted SSD for passenger cars. If the grade is 3% or steeper, the designer should consider using the SSD values presented in Table 4-2. For nearly all roads, the grade is traversed in both directions; therefore, the designer should generally not adjust the SSD values for upgrades. Exceptions are for one-way roads and divided roadways where each carriageway has an independent alignment.

Table 4-2: Stopping Sight Distance (Passenger Cars — Adjusted for Grades)

Design Speed (km/h)	Stopping Sight Distance (m)													
	Downgrades							Upgrades						
	(-3%)	(-4%)	(-5%)	(-6%)	(-7%)	(-8%)	(-9%)	(3%)	(4%)	(5%)	(6%)	(7%)	(8%)	(9%)
20	20	20	20	20	20	20	20	19	18	18	18	18	18	18
30	32	33	33	35	34	35	35	31	31	30	30	30	30	29
40	50	49	50	50	51	52	53	45	45	44	44	43	43	43
50	66	67	68	70	71	72	74	61	61	60	59	59	58	58
60	87	88	90	92	93	95	97	80	79	78	77	76	75	75
70	110	112	114	116	119	122	124	100	99	98	97	95	94	93
80	136	138	141	144	147	151	154	123	121	120	118	117	115	114
90	164	167	171	174	178	183	187	148	146	143	141	140	138	136
100	194	198	203	207	212	218	223	174	172	169	167	165	162	160
110	227	232	238	243	249	256	262	203	200	197	194	191	189	186
120	263	269	275	281	289	297	304	234	231	227	223	220	217	214
130	302	308	315	323	331	340	350	267	263	259	254	251	247	243
140	342	349	358	367	377	387	399	303	297	292	288	283	279	275

Notes:

1. *Calculated SSDs are not shown. Table values were developed using Equation* 4.2 *and rounding up to the next highest 1-m increment.*
2. *These SSD are for grades ≥ 3%. For grades less than 3%, no adjustment is necessary (i.e. use the level SSD values from Table* 4-1*).*
3. *For intermediate grades, use Equation* 4.2 *and round up to the next highest 1-m increment.*

4.3.3 Trucks

In general, trucks require longer SSD for a given speed than passenger vehicles due to their size and mass. Truck SSDs are determined based on the friction of the truck tire with the pavement surface and the driver efficiency (reaction time). Equation 4.3 is used to determine SSD values for trucks. The friction value (*f*) is a combination of numerous factors (e.g. weather, pavement condition, tire tread, truck braking efficiency). For design, a friction factor of 0.29 is used as a reasonable balance of all factors. Reaction times for experienced truck driver are generally better than those for drivers of passenger cars. Consequently, in constrained urban situations where drivers are alert, a reaction time of 2.0 seconds may be considered. For rural and unconstrained situations, a reaction time of 2.5 seconds should be used. SSD values for trucks are shown in Table 4-3 for level grade (i.e. less than 3%) and Table 4-4 for grades equal to greater than 3%. Similar to passenger cars, a height of 600 mm is used for the object on the road.

$$\mathbf{SSD} = \frac{\mathbf{Vt}}{\mathbf{3.6}} + \frac{\mathbf{V^2}}{\mathbf{254\,(f \pm G)}}$$

Equation 4.3: Truck SSD

where: SSD = stopping sight distance, m
t = reaction time, sec
V = design speed, km/h
f = friction factor, (assume 0.29)
G = longitudinal grade, decimal (+ for upgrades and – for downgrades)

Table 4-3: Stopping Sight Distance (Trucks — Level Grade)

Operating Speed (km/h)	t = 2.0 (sec)	t = 2.5 (sec)
40	50 m*	50 m*
50	65 m*	70 m
60	85 m*	95 m
70	105 m	115 m
80	130 m	145 m
90	160 m	175 m
100	190 m	205 m

* *Values have been adjusted up to match passenger car SSD from Table* 4-1.

Note: Only use the 2.0 seconds reaction time in constrained situations where drivers will be alert. For all other situations, use a reaction time of 2.5 seconds.

Source: (2)

Table 4-4: Grade Adjustments for Truck SSD

Design Speed (km/h)	Adjustments to Truck Stopping Sight Distance (m)													
	Downgrades							Upgrades						
	(-3%)	(-4%)	(-5%)	(-6%)	(-7%)	(-8%)	(-9%)	(3%)	(4%)	(5%)	(6%)	(7%)	(8%)	(9%)
40	3	3	5	6	7	8	10	-2	-3	-3	-4	-4	-5	-5
50	4	5	7	9	11	13	15	-3	-4	-5	-6	-7	-7	-8
60	6	8	10	13	16	19	22	-5	-6	-7	-8	-10	-11	-12
70	8	11	14	17	21	25	30	-6	-8	-10	-11	-13	-14	-16
80	10	14	18	23	28	33	39	-8	-11	-13	-15	-17	-19	-21
90	13	18	23	29	35	42	49	-10	-13	-16	-19	-21	-24	-26
100	16	22	28	35	43	52	61	-13	-16	-20	-23	-26	-29	-32

Notes:

1. *These adjustments are for grades ≥ 3%. The designer should adjust SSD values from Table* 4-3 *by adding or subtracting these grade corrections. Downgrades are shown as negative, with upgrades listed as positive.*

2. *For grades less than 3%, no adjustment is necessary (i.e. use the level SSD values from Table* 4-3*).*

4.3.4 SSD Application

The application of the SSD to a specific geometric element (e.g. horizontal sight distance, crest vertical curve) is discussed in the applicable chapters of the Abu Dhabi *Road Geometric Design Manual* (e.g. Chapter 5 "Horizontal Alignment," Chapter 6 "Vertical Alignment").

4.4 Decision Sight Distance

4.4.1 Theoretical Discussion

Drivers may be required to make decisions where the roadway environment is difficult to perceive or where unexpected manoeuvres are required. These are areas of concentrated demand where the roadway elements, traffic volumes, and traffic control devices may all compete for the driver's attention. This relatively complex environment may increase the required driver perception/reaction time beyond that provided by the SSD values (2.5 seconds) and, in some locations, the desired vehicular manoeuvre may be a speed/path/ direction change rather than a stop. At these locations, the designer should consider providing decision sight distance to provide an additional margin of safety. The various avoidance manoeuvres for decision sight distance are:

- Avoidance Manoeuvre A: Stop on rural road. (t = 3.0 s)
- Avoidance Manoeuvre B: Stop on urban road. (t = 9.1 s)
- Avoidance Manoeuvre C: Speed/path/direction change on rural road (t = 10.2 – 11.2 s)
- Avoidance Manoeuvre D: Speed/path/direction change on suburban road.(t = 12.1 – 12.9 s)
- Avoidance Manoeuvre E: Speed/path/direction change on urban road. (t = 14.0 – 14.5 s)

The decision sight distances for avoidance manoeuvres A and B are determined using Equation 4.4.

$$\mathbf{DSD = 0.278Vt + 0.039\frac{V^2}{a}}$$

Equation 4.4: DSD Manoeuvres A and B

where: DSD = decision sight distance, m
t = pre-manoeuvre time, sec (see Section 4.4.1)
V = design speed, km/h
a = driver deceleration, m/s^2 (3.4 m/s^2)

The decision sight distances for avoidance manoeuvres C, D, and E are determined using Equation 4.5.

$$\mathbf{DSD = 0.278Vt}$$

Equation 4.5: DSD Manoeuvres C, D and E

where: DSD = decision sight distance, m
t = pre-manoeuvre time, sec (see Section 4.4.1)
V = design speed, km/h

Table 4-5 provides the decision sight distance for each manoeuvre based on the design speed.

Table 4-5: Decision Sight Distance

Design Speed (km/h)	Decision Sight Distance (m)				
	Avoidance Manoeuvre				
	A	B	C	D	E
50	70	155	145	170	195
60	95	195	170	205	235
70	115	235	200	235	275
80	140	280	230	270	315
90	170	325	270	315	360
100	200	370	315	355	400
110	235	420	330	380	430
120	265	470	360	415	470
130	305	525	390	450	510
140*	345	580	420	485	550

Notes:

1. *Avoidance Manoeuvre A: Stop on rural road.*
2. *Avoidance Manoeuvre B: Stop on urban road.*
3. *Avoidance Manoeuvre C: Speed/path/direction change on rural road.*
4. *Avoidance Manoeuvre D: Speed/path/direction change on suburban road.*
5. *Avoidance Manoeuvre E: Speed/path/direction change on urban road.*

* *140 km/h values estimated by DMT.*

Source: (1)

4.4.2 Applications

In general, the designer should consider using decision sight distance at any relatively complex location where the driver perception/reaction time may exceed 2.5 seconds. Example locations where decision sight distance may be appropriate include:

- freeway exit/entrance gores;
- freeway lane drops;
- freeway left-side entrances or exits;
- intersections near a horizontal curve;
- roadway/railroad grade crossings;
- approaches to detours and lane closures;
- along high-speed, high-volume urban arterials with considerable roadside friction; or
- isolated traffic signals on high-speed, rural roadways.

As with SSD, the driver height of eye is 1080 mm and the height of object is typically 600 mm. However, potential sites for decision sight distance may also be sites where the object is assumed to be the pavement surface (e.g., freeway exit gores). Therefore, the designer may decide that a 0.0-mm height of object for application is more appropriate at some sites.

4.5 Passing Sight Distance

Passing sight distance considerations are limited to two-way, single carriageways. On these facilities, vehicles may overtake slower moving vehicles and the passing manoeuvre must be accomplished on a lane used by the opposing traffic.

If passing is to be accomplished without interfering with an opposing vehicle, the passing driver should be able to see a sufficient distance ahead, clear of traffic, so that the passing driver can decide whether to initiate and to complete the passing manoeuvre safely before meeting an opposing vehicle that appears during the manoeuvre.

Passing sight distance for use in design should be based on a single passenger vehicle passing a single passenger vehicle. While there may be occasions to consider where two or more vehicles pass or are passed, it is not practical to assume such conditions in developing minimum design criteria. Longer sight distances occur in design and these locations can accommodate an occasional multiple passing manoeuvre or passing a truck.

Significant grades may affect the sight distance needed for passing. However, if the passing vehicle is on an upgrade and the opposing vehicle is on a downgrade (or vice versa), the effects of the grade on acceleration capabilities to the vehicles may be offset. Most passing drivers use sound judgement about whether to initiate and complete a pass with significant grades involved with the manoeuvre.

Minimum passing sight distances for use in design are based on the minimum sight distances for marking no-passing zones on two-lane roads as presented in the United States *Manual on Uniform Traffic Control Devices* (35). The height of eye and height of object for passing sight distance should be 1.080 m. Table 4-6 provides passing sight distances based on design speed.

Table 4-6: Passing Sight Distances for Two-Way Single Carriageway

Design Speed (km/h)	Passing Sight Distance (m)
30	120
40	140
50	160
60	180
70	210
80	245
90	280
100	320
110	355
120	395

Source: (1) (35)

The development of the minimum passing sight distances for design incorporate certain assumptions about the driver's behaviour. The designer should note the following:

- The speeds of the passing and opposing vehicles are equal and are travelling at the design speed of the road.
- The passed vehicle travels at a uniform speed and the speed differential between the passing and passed vehicles is 19 km/h.
- The passing vehicle has sufficient acceleration capability to reach the specified speed differential relative to the passed vehicle by the time it reaches the critical position, which generally occurs about 40% of the way through the passing manoeuvre.
- A vehicle length of 5.8 m is used for both the passing and passed vehicles.
- The passing driver's perception-reaction time in deciding to abort the passing manoeuvre is 1 s.
- If a passing manoeuvre is aborted, the passing vehicle will use a deceleration rate of 3.4 m/s^2, the same deceleration rate used in stopping sight distance criteria.
- For a completed or aborted pass, the space headway between the passing and passed vehicles is 1 s.
- There is a minimum 1 s headway between an opposing vehicle and the passing vehicle after the passing vehicle has completed the passing manoeuvre.

Guidance on determining passing zone locations is discussed in Section 8.2.5.2.

4.6 Intersection Sight Distance

4.6.1 General

For an at-grade intersection to operate properly, adequate sight distance should be available for a driver to perceive potential conflicts and to perform the actions needed to negotiate the intersection safely. The additional costs and impacts of removing sight obstructions are often justified. If it is impractical to remove an obstruction blocking the sight distance, the designer should consider providing traffic control devices or design applications (e.g. warning signs, turn lanes) that may not otherwise be considered.

In general, intersection sight distance (ISD) refers to the corner sight distance available in intersection quadrants that allows a driver approaching an intersection to observe the actions of vehicles on the crossing leg(s). ISD evaluations involve establishing the needed sight triangle in each quadrant by determining the legs of the triangle on the two crossing roadways. Within this clear sight triangle, if practical, remove or lower any object that would obstruct the driver's view. These objects may include buildings, parked or turning vehicles, trees, hedges, plantings, signs, fences, retaining walls, and the actual ground line. In addition, where an interchange ramp or crossroad intersects the major road near a bridge on a crest vertical curve, items such as bridge parapets, piers, abutments, guardrail, or the crest vertical curve itself may restrict the clear sight triangle.

The necessary clear sight triangle is based on the type of traffic control at the intersection and on the design speeds of the two roadways. The types of traffic control and manoeuvres are as follows:

- Case A – Intersections with no control (*not applicable in Abu Dhabi*):
- Case B – Intersections with stop control on the minor road:
  - Case B1 – Left-turn from the minor road,
  - Case B2 – Right–turn from minor road,
  - Case B3 – Crossing manoeuvre from the minor road;
- Case C – Intersections with yield control on the minor road:
  - Case C1 – Crossing manoeuvre from the minor road,
  - Case C2 – Left or right turn from the minor road;
- Case D – Intersections with traffic signal control;
- Case E – Intersections with all-way stop control; and
- Case F – Left turns from the major road.

4.6.2 Eye and Object Heights

For applicable eye and object heights, see Section 4.2.

4.6.3 Case A – Intersections with No Control

Intersections between low-volume and low-speed roads may have no traffic control (e.g. no STOP or YIELD sign). This particular case is not applicable in Abu Dhabi.

4.6.4 Case B – Intersections with Stop Control on the Minor Road

Where traffic on the minor road of an intersection is controlled by STOP signs, the driver of the vehicle on the minor road must have sufficient sight distance for a safe departure from the stopped position assuming that the approaching vehicle comes into view as the stopped vehicle begins its departure.

4.6.4.1 Basic Design Criteria

Gap acceptance is used as the conceptual basis for ISD design criteria. The ISD methodology is intended to find a balance between an acceptable level of safety and what can be provided at an intersection on a practical basis. It ensures that an intersection operates smoothly without forcing a vehicle on the major road to stop. As the crossroad vehicle makes the turn and accelerates, field studies have indicated that mainline vehicles reduce their speed to approximately 70% of the mainline design speed to compensate for the entering vehicle.

The ISD is obtained by providing clear sight triangles to the left and right as shown in Figure 4-1 and Figure 4-2, respectively. The lengths of legs of these sight triangles are determined as follows:

Figure 4-1: Clear Sight Triangle for Viewing Traffic Approaching From Left

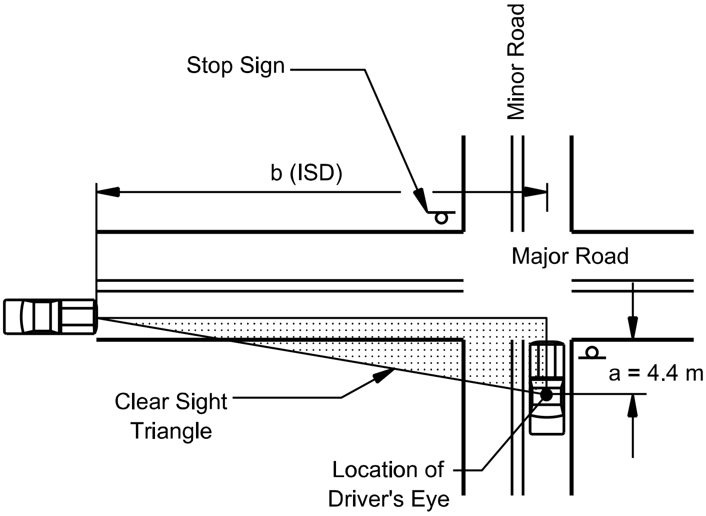
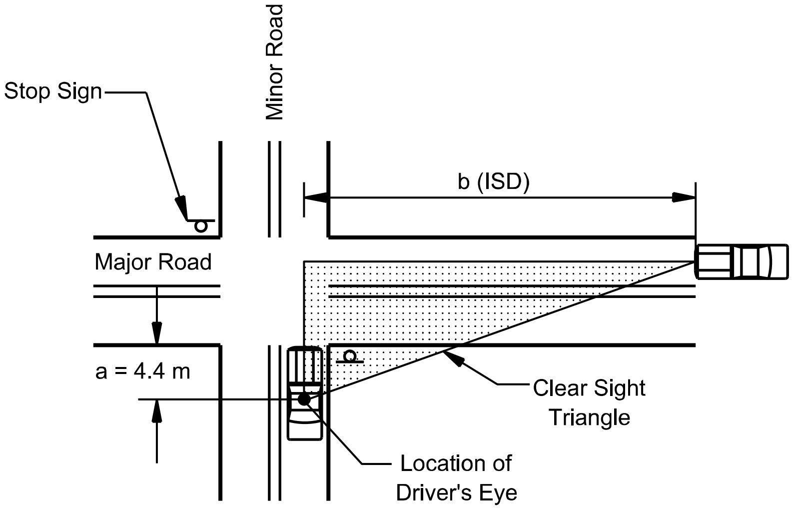


Figure 4-2: Clear Sight Triangle for Viewing Traffic Approaching From Right



1. Minor Road. The length of leg along the minor road is based on two parts. The first is the location of the driver's eye on the minor road. This is typically assumed to be 4.4 m from the edge of the major road travelled way for a stopped vehicle. The second part is based on the distance to the centre of the vehicle on the major road. For right-turning vehicles, this is assumed the centre of the closest travel lane from the left. For left-turning vehicles, this is assumed to be the centre of the closest travel lane for vehicles approaching from the right.

2. Major Road. The length of the sight triangle or ISD along the major road is determined using Equation 4.6.

$$\mathbf{b = ISD = 0.278\ V_{major}\ t_c}$$

Equation 4.6: ISD Basic Equation

where: b = length of sight triangle along the major road or ISD, m

V_{major} = design speed of major road, km/h

t_c = critical gap for entering or crossing the major road, sec

The critical gap time (t_c) varies according to the design vehicle, the grade on the minor road approach, the number of lanes on the major roadway, the type of operation and the intersection skew.

4.6.4.2 Case B1 – Left Turns Onto Major Roadway

To determine the intersection sight distance for vehicles turning left onto the major road, the designer should use Equation 4.6 and the gap times (t_c) presented in Table 4-7. Table 4-7 also presents adjustments to the gap times for multilane facilities and steep grades on the minor road approach. These adjustments are further discussed below. Table 4-8 provides the ISD values for typical design vehicles on two-lane, level facilities. The designer should consider the following:

1. Multilane Facilities. For multilane facilities, the gap acceptance times presented in Table 4-7 may need to be adjusted to account for the additional distance required by the left turning vehicle to cross the additional lanes or median. For left turns onto multilane roadways without a median, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane from the left, in excess of one, to be crossed by the turning vehicle. Assume that the left-turning driver will enter the left most travel lane on the far side of the major road.

Table 4-7: Gap Acceptance Times – Stopped Controlled (Left Turns From Minor Road)

Design Vehicle	Gap Acceptance Time (t_c) (sec)
Passenger Car	7.5
Single-Unit Truck	9.5
Tractor/Semitrailer	11.5

Note: Times are for left turns onto a two-lane roadway without a median and may require adjustments to the base time gap.

Adjustments:

1. *Multilane Roadways. For left turns onto two-way multilane roadways without a median, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane from the left, in excess of one, to be crossed by the turning vehicle. See discussion in Section 4.6.4.2 for additional guidance.*

2. *Minor Road Approach Grades. If the approach grade on the minor road exceeds +3%, multiple 0.2 seconds times the actual percent grade on the minor approach and add this number to the base time gap.*

3. *Major Road Approach Grade. Major road grade does not affect calculations.*

Source: (1)

Table 4-8: Intersection Sight Distances for Two-Way Single Carriageway (Left Turns From Minor Road)

Design Speed (V_{major}) (km/h)	Passenger Cars (m)	Single-Unit Trucks (m)	Tractor/ Semitrailers (m)
30	63	80	96
40	84	106	128
50	105	133	160
60	126	159	192
70	146	185	224
80	167	212	256
90	188	238	288
100	209	265	320
110	230	291	352
120	251	317	384

Notes:

1. *These ISD values assume left turns onto a two-lane facility without a median.*
2. *These ISD values assume a minor road approach grade $\leq$+3%.*

2. Left Turns Through Medians.

   a. Narrow Medians. For a facility that does not have a median wide enough to store a stopped design vehicle, divide the median width by 3.65 m (for urban areas the lane width may be lesser as shown in Chapter 9) to get the corresponding number of lanes and then use the criteria in Item 1 above to determine the additional time factor.

   b. Wide Medians. For a facility that does have a median wide enough to store a stopped design vehicle, the designer should evaluate the sight distance needed in two separate steps:

      - First, with the design vehicle stopped on the side road, use the gap acceptance times for a vehicle turning right. See Section 4.6.4.3 for right turning vehicles to determine the applicable ISD. Under some circumstances, it also may be necessary to check the straight through crossing manoeuvre to determine if it is the critical movement. Straight through crossing criteria are discussed in Section 4.6.4.4.

      - Second, with the design vehicle stopped in the median, assume a two-lane roadway design and use the gap acceptance times for a vehicle turning left or use Table 4-8 directly to determine the applicable ISD.

3. Approach Grades. If the approach grade on the minor road exceeds 3%, see the criteria in Table 4-7.

4. Trucks. At some intersections (e.g. near truck stops, interchange ramps), the designer may want to use the truck as the design vehicle for determining the ISD. The gap acceptance times (t_c) for single-unit and tractor/semitrailer trucks are provided in Table 4-7. Calculated ISD values for two-lane roads are presented in Table 4-8. The height of eye for these vehicles is discussed in Section 4.2.

4.6.4.3 Case B2 – Right Turns Onto Major Roadway

To determine the intersection sight distance for vehicles turning right onto the major road, the designer should use Equation 4.6 and the gap times (t_c) presented in Table 4-9. Because the turning vehicle is assumed to be turning into the nearest right through lane, no adjustments to the gap times are required. This is the same for either two-lane or multilane roads.

Table 4-9 also presents adjustments to the gap times for steep grades on the minor road approach. Table 4-10 provides the ISD values for typical design vehicles on two-lane, level carriageways.

4.6.4.4 Case B3 – Crossing Manoeuvre from the Minor Road

In the majority of cases, the intersection sight distance for a crossing manoeuvre is less than that required for a left-turning vehicle. However, in the following situations, the straight through crossing sight distance may be the more critical movement:

- where left turns are not permitted from a particular approach and the crossing manoeuvre is the only legal or expected movement (e.g. indirect left turns);
- where the design vehicle must cross more than four travel lanes or with medians, the equivalent distance; or
- where substantial volumes of heavy vehicles cross the roadway and there are steep grades on the minor road approaches.

Use Equation 4.6 and the gap acceptance times (t_c) and adjustment factors in Table 4-9 to determine the ISD for crossing manoeuvres. Where narrow medians are present which cannot store the design vehicle, include the median width in the overall width to determine the applicable gap time. Divide this overall width by 3.65 m (for urban areas the lane width may be lesser as shown in Chapter 9) to determine the corresponding number of lanes for the crossing manoeuvre. Add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane, in excess of two, to be crossed by the design vehicle.

4.6.5 Case C – Intersections with Yield Control on the Minor Road

At intersections controlled by a yield sign, drivers on the minor road will typically:

- slow down as they approach the major road, typically to 60% of the approach speed;
- based on their view of the major road, make a stop or continue decision; and
- brake to a stop, continue their crossing, or turn onto the major road.

Yield control criteria are based on a combination of the no control ISD and the stop-controlled ISD as discussed in Section 4.6.4. To determine the applicable clear sight triangles of the approaches, the following Sections will apply.

Table 4-9: Gap Acceptance Times – Stopped Controlled (Right Turns From Minor Road/Crossing Manoeuvre)

Design Vehicle	Gap Acceptance Time (t_c) (sec)
Passenger Car	6.5
Single-Unit Truck	8.5
Tractor/Semitrailer	10.5

Adjustments:

1. *Minor Road Approach Grades. If the approach grade on the minor road exceeds +3%, multiply 0.1 seconds times the actual percent grade on the minor road approach and add this number to the base time gap.*

2. *Major Road Approach Grade. Major road grade does not affect calculations.*

Source: (1)

Table 4-10: Intersection Sight Distances For Two-Way Single Carriageway (Right Turns From Minor Road/Crossing Manoeuvre)

Design Speed (V_{major}) (km/h)	Passenger Cars (m)	Single-Unit Trucks (m)	Tractor/ Semitrailers (m)
30	55	71	88
40	73	95	117
50	91	119	146
60	109	142	176
70	127	166	205
80	145	190	234
90	163	213	263
100	181	237	292
110	199	260	322
120	217	284	351

Note: These ISD values assume a minor road approach grade $\leq$ +3%. If the approach grade on the minor road exceeds 3%, see the criteria in Table 4-9*.*

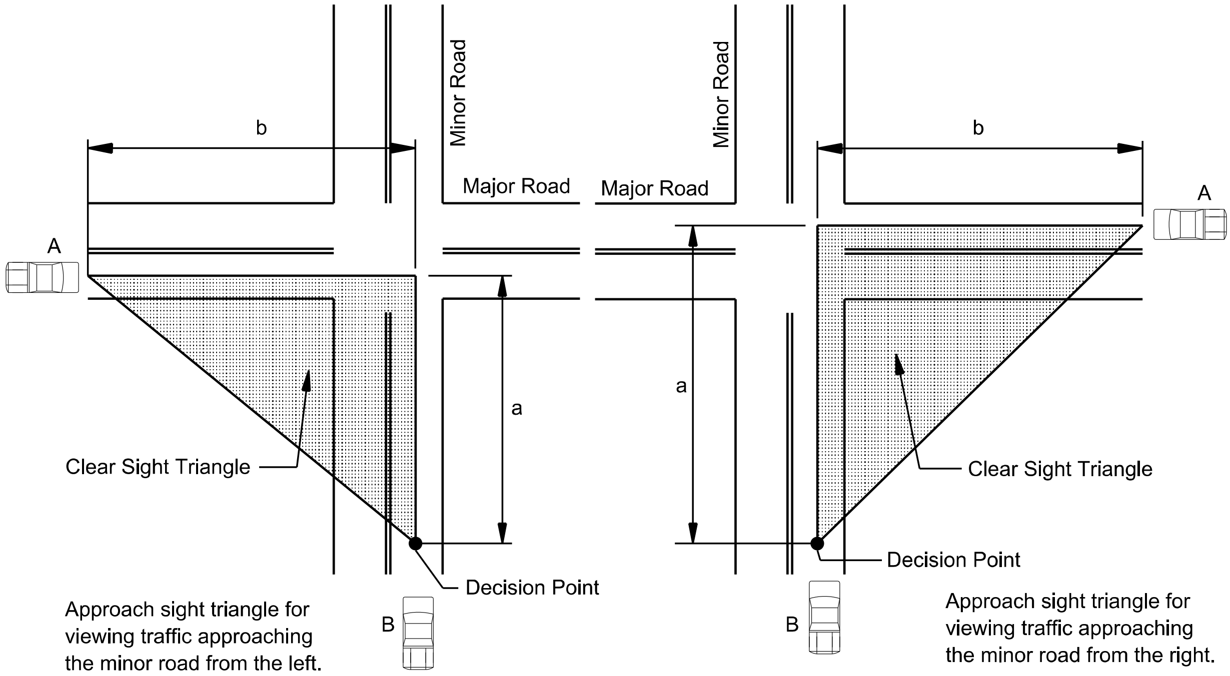
4.6.5.1 Case C1 – Crossing Manoeuvre from the Minor Road

Use the following to determine the legs of the clear sight triangle; see Figure 4-3:

1. Minor Road. The leg on the minor road approach can be determined directly from Table 4-11.

2. Major Road. The leg on the major road is determined using the following equations and the times listed in Table 4-11.

Figure 4-3: Approach Sight Triangles (Yield Control)



Source: (1)

Table 4-11: ISD Yield Controlled Intersection (Crossing Manoeuvre)

Minor Road Approach			Travel Time (t_c) (sec)	
Design Speed (km/h)	**Length of Leg(1) (m)**	**Travel Time (t_a) (1,2) (sec)**	**Calculated Value**	**Design Value(3,4)**
20	20	3.2	7.1	7.1
30	30	3.6	6.2	6.5
40	40	4.0	6.0	6.5
50	55	4.4	6.0	6.5
60	65	4.8	6.1	6.5
70	80	5.1	6.2	6.5

Notes:

1. *For minor road approach grades that exceed 3%, multiply the distance or the time in this table by the appropriate adjustment factor from Table* 4-12*.*
2. *Travel time applies to a vehicle that slows before crossing the intersection, but does not stop.*
3. *The value of t_c should equal or exceed the appropriate critical gap for crossing the major road from a stop-controlled approach.*
4. *Values shown are for passenger cars crossing a two-lane roadway with no median and grades 3% or less.*

Source: (1)

Table 4-12: Adjustment Factors for Approach Grade

Approach Grade (%)	Design Speed (km/h)								
	20	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**
-6	1.1	1.1	1.1	1.1	1.1	1.1	1.2	1.2	1.2
-5	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
-4	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1
-3 to +3	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
+4	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9
+5	1.0	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9
+6	1.0	1.0	0.9	0.9	0.9	0.9	0.9	0.9	0.9

Source: (1)

$$t_c = t_a + \frac{w + L_a}{0.167\left(V_{min\ or}\right)}$$

Equation 4.7 Crossing Time

$$b = ISD = 0.278\ V_{major}\ t_c$$

Equation 4.8 Yield ISD

where: b = length of leg of sight triangle or ISD along the major road, m

t_c = travel time to reach and clear the major road in a crossing manoeuvre, sec

t_a = travel time to reach the major road from the decision point for a vehicle that does not stop, sec (use appropriate value for the minor-road design speed from Table 4-11, adjusted for approach grade, where appropriate)

w = width of intersection to be crossed, m

L_a = length of design vehicle, m

V_{major} = design speed of major road, km/h

V_{minor} = design speed of minor road, km/h

4.6.5.2 Case C2 – Left or Right Turn From the Minor Road

For the left- or right-turning vehicle, the approach legs are determined as follows (see Figure 4-3):

1. Minor Road. The assumed turning speed from the minor road to the major road is 16 km/h. This corresponds to an approach distance of 25 m along the minor road leg.
2. Major Road. To determine the legs along the major road, use the same procedures as discussed in Section 4.6.4 for the stop-controlled intersection, Equation 4.7 and the critical gap times listed in Table 4-13. Because the critical gap times are longer than the stop-controlled gap times, it will be unnecessary to determine the sight distance criteria for the vehicle that stops at the yield sign.
3. Left Turns. For left turns onto two-way roadways with more than two lanes, add 0.5 s for passenger cars or 0.7 s for trucks for each additional lane, from the left, in excess of one, to be crossed by the turning vehicle.
4. Right Turns. For right turns, no adjustment is necessary.

Table 4-13: Gap Acceptance Times (Left and Right Turns From Minor Road)

Design Vehicle	Gap Acceptance Time (t_c) (sec)
Passenger car	8.0
Single-unit truck	10.0
Tractor/semitrailer	12.0

Note: Time gaps are for a vehicle to turn right or left onto a two-lane roadway with no median.

Source: (1)

4.6.6 Case D – Intersections with Traffic Signal Control

At signalised intersections, provide sufficient sight distance so that the first vehicle on each approach is visible to all other approaches. Traffic signals are often used at high-volume intersections to address crashes related to restricted sight distances. Therefore, the ISD criteria for left- or right-turning vehicles as discussed in Section 4.6.4 is typically not applicable at signalised intersections.

However, where right-turn-on-red is allowed, check to see that the ISD as presented in Section 4.6.4.3 for a stop-controlled right-turning vehicle is available to the left. If it is not, this may warrant restricting the right-turn-on-red movement. In addition, if the traffic signal is placed on two-way flash operation (i.e. flashing amber on the major-road approaches and flashing red on the minor-road approaches) under off-peak or night time conditions, provide the ISD criteria as discussed in Section 4.6.4 for a stop-controlled intersection.

4.6.7 Case E—Intersections with All-Way Stop Control

At intersections with all-way stop control, provide enough sight distance so that the first stopped vehicle on each approach is visible to all the other approaches. The ISD criteria for left- or right-turning vehicles as discussed in Section 4.6.4 are not applicable in this situation. Often, intersections are converted to all-way stop control to address limited sight distance at the intersection. Therefore, providing additional sight distance at the intersection is unnecessary.

4.6.8 Case F – Left Turns From the Major Road

At all intersections, regardless of the type of traffic control, the designer should consider the sight distance needs for a stopped vehicle turning left from the major road. This situation is illustrated in Figure 4-4. The driver will need to see straight ahead for a sufficient distance to turn left and clear the opposing travel lanes before an approaching vehicle reaches the intersection. In general, if the major roadway has been designed to meet the stopping sight distance criteria, intersection sight distance only will be a concern where the major road is on a horizontal curve, where there is a median, or where there are opposing vehicles making left turns at an intersection. Sight distance for opposing left turns may be increased by offsetting the left-turn lanes.

Use Equation 4.6 and the gap acceptance times (t_c) from Table 4-14 to determine the applicable intersection sight distances for the left-turning vehicle. Where the left-turning vehicle must cross more than one opposing lane, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane in excess of one. Where medians are present and the left-turn lanes are not offset, the designer will need to consider the median width in the same manner as discussed in Section 4.6.4. Table 4-15 provides the ISD values for typical design vehicles turning across one or two lanes.

Table 4-14: Gap Acceptance Times (Left Turns From Major Road)

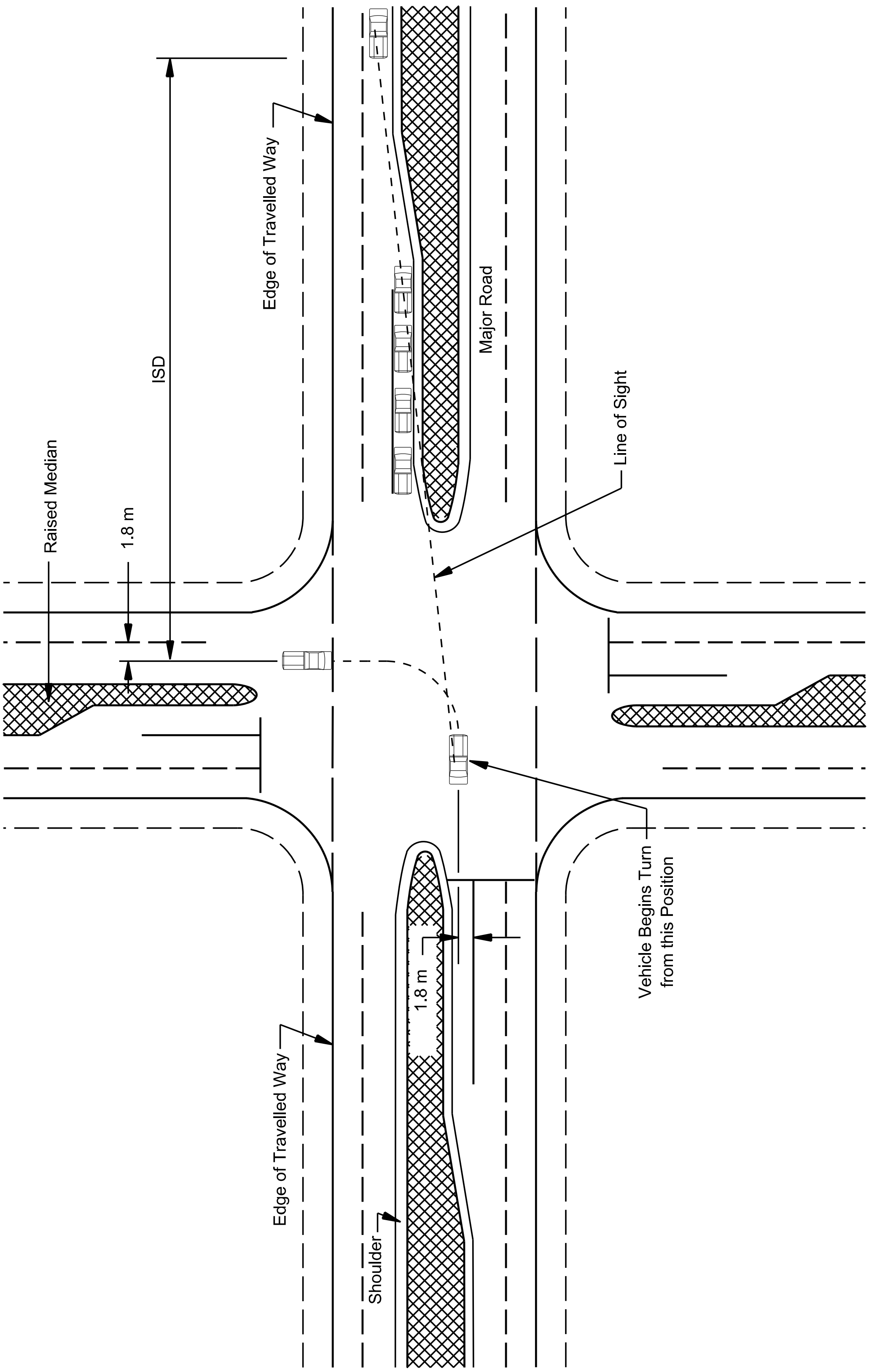
Design Vehicle	Gap Acceptance Time (t_c) (sec)
Passenger Car	5.5
Single-Unit Truck	6.5
Tractor/Semitrailer	7.5

Adjustments:

Where left-turning vehicles cross more than one opposing lane, add 0.5 seconds for passenger cars or 0.7 seconds for trucks for each additional lane in excess of one. See Section 4.6.8 for additional guidance on median widths.

Source: (1)

Figure 4-4: Intersection Sight Distance (Left Turns From the Major Road)



Note: See Section 4.6.8 for discussion and application.

Table 4-15: Intersection Sight Distance (Left Turns From Major Road)

Design Speed (V_{major}) (km/h)	ISD					
	Passenger Cars		Single-Unit Trucks		Tractor/Semitrailers	
	Crossing 1 Lane (m)	Crossing 2 Lanes (m)	Crossing 1 Lane (m)	Crossing 2 Lanes (m)	Crossing 1 Lane (m)	Crossing 2 Lanes (m)
30	46	50	55	60	63	69
40	62	67	73	81	84	92
50	77	84	91	101	105	114
60	92	101	109	121	126	137
70	107	117	127	141	146	160
80	123	134	145	161	167	183
90	138	151	163	181	188	206
100	153	167	181	201	209	228
110	169	184	199	221	230	251
120	184	201	217	241	251	274

Note: *Assumes no median on major road.*

Source: (1)

4.6.9 Examples of ISD Applications

The following three examples illustrate the application of the ISD criteria:

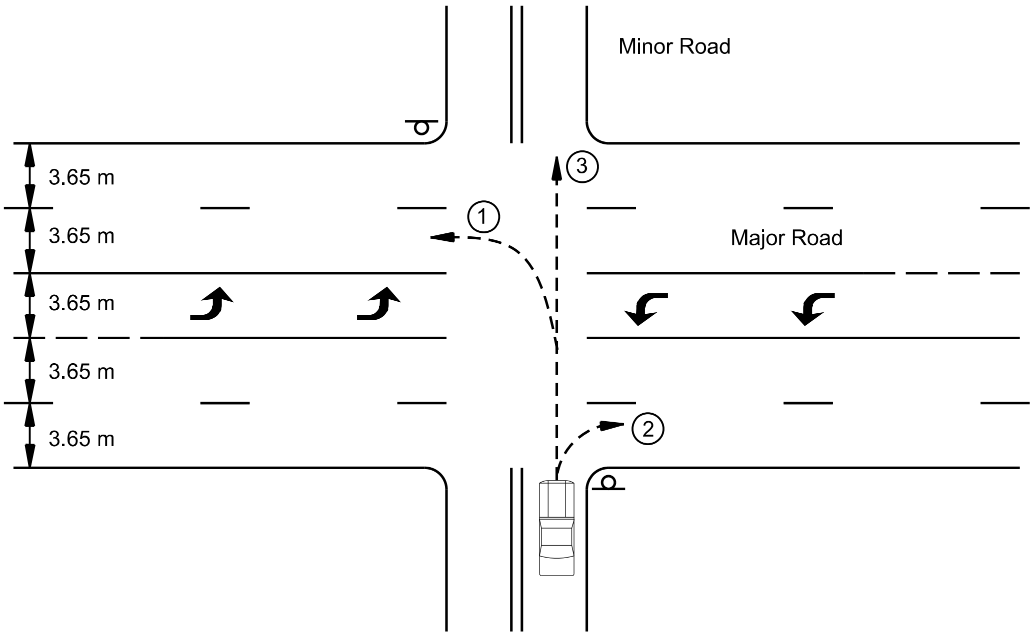
Example 4.6–1

Given: Minor road intersects a four-lane roadway with a left-turn lane.
Minor road is stop controlled and intersects major road at 90°.
Design speed of the major roadway is 70 km/h.
All travel lane widths are 3.65 m.
The left turn lane width is 3.65 m.
Grade on minor road is 1%.
Trucks are not a concern.
See Figure 4-5.

It is to be noted that the lane width used here is for illustration purposes and that for urban areas the lane width may be lesser as shown in Chapter 9.

Problem: Determine the intersection sight distance needed to the left and right of the minor road. See Figure 4-1 and Figure 4-2 respectively.

Figure 4-5: Example 4.6 – 1



Solution:

1. For the passenger car turning left, the ISD to the right must reflect the additional time required to cross the additional lanes and left-turn lane; see Section 4.6.4.2. The following will apply:

   a. First, determine the extra width required by the one additional travel lane and the left-turn lane and divide this number by 3.65 m:

   $$\frac{(3.65 + 3.65)}{3.65} = 2 \text{ lanes}$$

   b. Next, multiply the number of lanes by 0.5 seconds to determine the additional time required:

   (2 lanes)(0.5 sec/lane) = 1.0 second

   c. Add the additional time to the basic gap time of 7.5 seconds and insert this value into Equation 4.6:

   ISD = (0.278)(70)(7.5 + 1.0) = 166 m

Provide an ISD of 166 m to the right for the left-turning vehicle.

2. For the passenger car turning right, the ISD to the left can be determined directly from Table 4-10, because the right-turning motorist is assumed to turn into the near lane. For the 70 km/h design speed, the ISD to the left is 127 m.

3. Check the passenger vehicle crossing the mainline, as discussed in Section 4.6.4.4. The following will apply:

a. First determine the extra width required by the two additional travel lanes and the left-turn lane and divide this number by 3.65 m:

$$\frac{(3.65 + 3.65 + 3.65)}{3.65} = 3.0 \text{ lanes}$$

b. Next, multiply the number of lanes by 0.5 seconds to determine the additional time required:

(3.0 lanes)(0.5 sec/lane) = 1.5 seconds

c. Add the additional time to the basic gap time of 6.5 seconds and insert this value into Equation 4.6:

ISD = (0.278)(70)(6.5 + 1.5) = 156 m

The 156 m for the crossing manoeuvre is less than the 166 m required for the left-turning vehicle and, therefore, is not the critical manoeuvre.

4. Prepare a scaled drawing in the horizontal and vertical planes and graphically check to determine if the applicable ISD is available.

<u>Example 4.6 – 2</u>

Given: Rural minor road intersects a four-lane divided roadway.
Minor road is stop controlled and intersects major road at 90°.
Design speed of the major roadway is 100 km/h.
All travel lane widths are 3.65 m.
The median width is 18.00 m.
Grade on minor road is +2%.
The design vehicle is a 64-passenger school bus that is 10.90 m long.
For a school bus, assume a SU truck design vehicle for gap times.
See Figure 4-7

It is to be noted that the lane width used here is for illustration purposes and that for urban areas the lane width may be lesser as shown in Chapter 9.

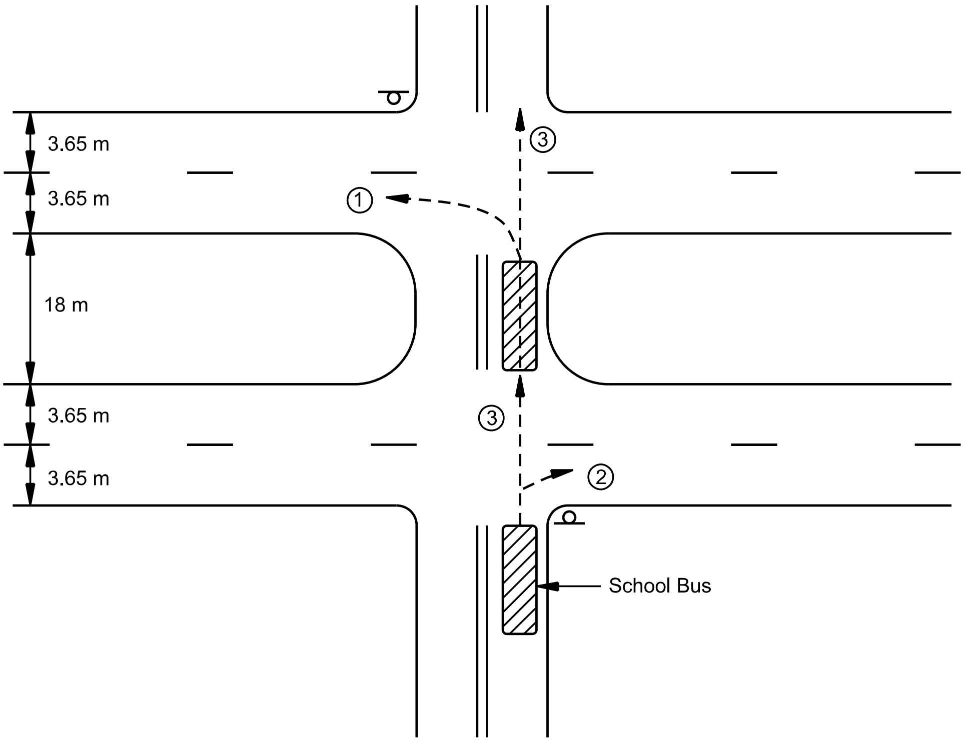
Problem: Determine the intersection sight distance needed to the left and right of the minor road. See Figure 4-1 and Figure 4-2 respectively.

Solution:

1. For the school bus turning left, it can be assumed the school bus can safely stop in the median (i.e. 18.00 m minus 10.90 m). The ISD to the right can be determined directly from Table 4-8. For the 100 km/h design speed, the ISD to the right for the left turn is 265 m.

2. For the school bus turning right, the ISD to the left can be determined directly from Table 4-10. For the 100 km/h design speed, the ISD to the left is 237 m.

Figure 4-6: Example 4.6 – 2



3. Determine if the straight through crossing manoeuvre is critical; see Section 4.6.4.4. No adjustments are required to the base time of 8.5 seconds. Therefore, use Table 4-10 directly or Equation 4.6:

   ISD = (0.278)(100)(8.5) = 237 m

The crossing manoeuvre ISD is less than the left turning manoeuvre and the same as the right-turning manoeuvre and, therefore, is not critical.

4. Prepare a scaled drawing in the horizontal and vertical planes and graphically check to determine if the applicable ISD is available.

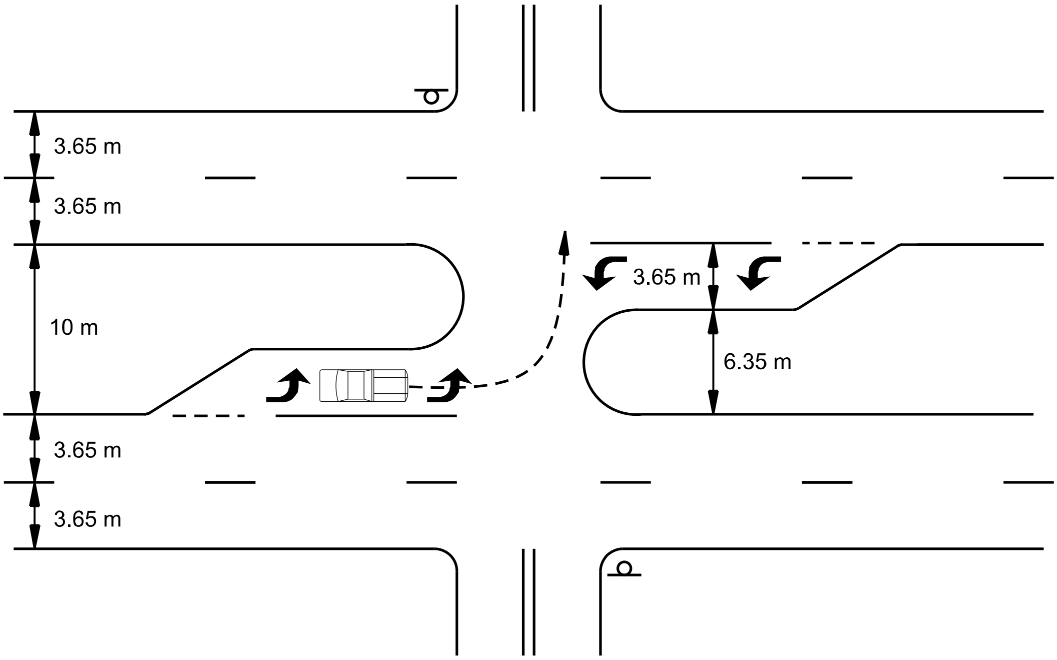
<u>Example 4.6 – 3</u>

Given: Rural minor road intersects a four-lane divided roadway.
Minor road is stop controlled and intersects major road at 90°.
Design speed of the major roadway is 80 km/h.
All travel lane widths are 3.65 m.
Existing median width is 10 m.
Trucks are not a concern.
See Figure 4-7.

It is to be noted that the lane width used here is for illustration purposes and that for urban areas the lane width may be lesser as shown in Chapter 9.

Problem: Determine the intersection design and sight distance for a vehicle turning left from the major road.

Figure 4-7: Example 4.6 – 3



Solution:

1. The existing 10 m wide median can accompany a 3.65 m wide left-turn lane, on the right side of the median. An adjustment is necessary for the remaining width of the left side of the median in computing the gap acceptance time (i.e. 10.00 m minus 3.65 m = 6.35 m).

2. For the passenger car turning left, the ISD to the right must reflect the additional time required to cross the median and an additional lane; see in Section 4.6.8. The following will apply:

   a. First, determine the extra width required by the median and one additional travel lane and divide this number by 3.65 m:

   $$\frac{(6.35 + 3.65)}{3.65} = 2.74 \text{ lanes}$$

   b. Next, multiply the number of lanes by 0.5 seconds to determine the additional time required:

   (2.74 lanes)(0.5 sec/lane) = 1.37 seconds

   c. Add the additional time to the basic gap time of 5.5 seconds and insert this value into Equation 4.6:

   ISD = (0.278)(80)(5.5 + 1.37) = 153 m

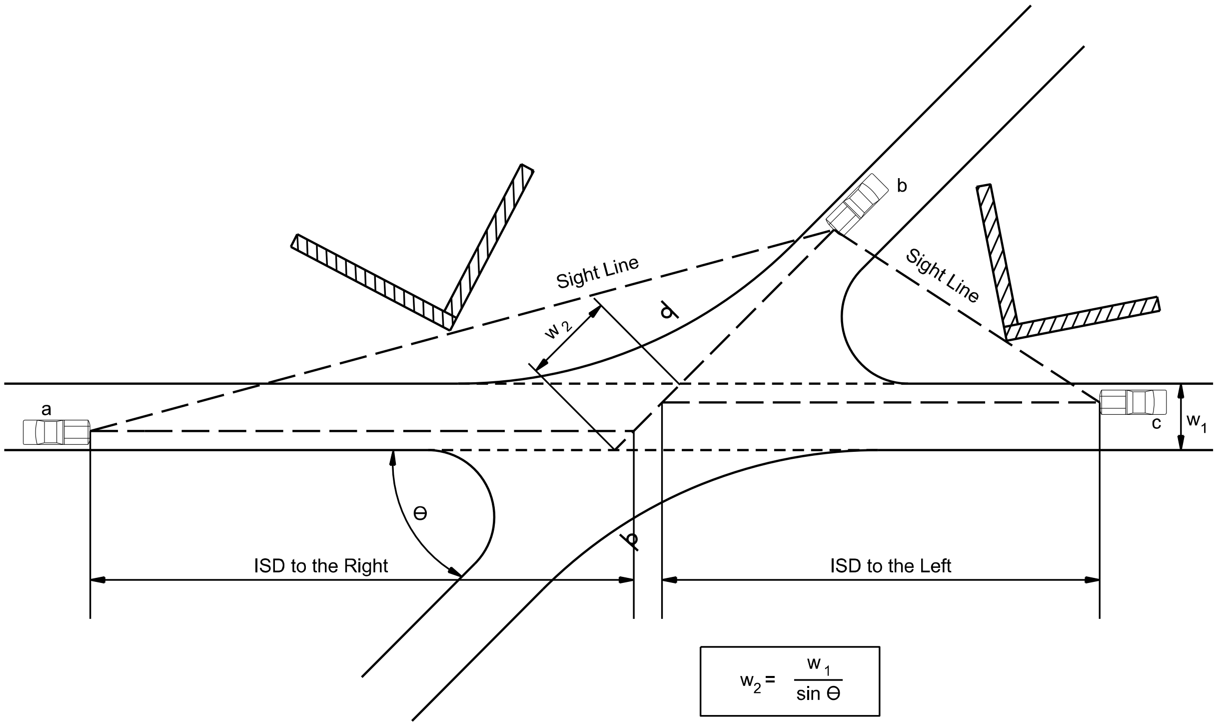
   Provide an ISD of 153 m ahead for the left-turning vehicle.

Prepare a scaled drawing in the horizontal and vertical planes and graphically check to determine if the applicable ISD is available.

4.6.10 Effect of Skew

Where is it is impractical to realign an intersection that is greater than 30 degrees from perpendicular, adjust the gap acceptance times presented in the above sections to account for the additional travel time required for a vehicle to make a turn or cross a facility. At oblique-angled intersections, determine the actual path length for a turning or crossing vehicle by dividing the total distance of the lanes and/or median to be crossed by the sine of the intersection angle. If the actual path length exceeds the total width of the lanes to be crossed by 3.65 m or more, apply the applicable adjustment factors; see Figure 4-8.

Figure 4-8: Sight Distance at Skewed Intersections

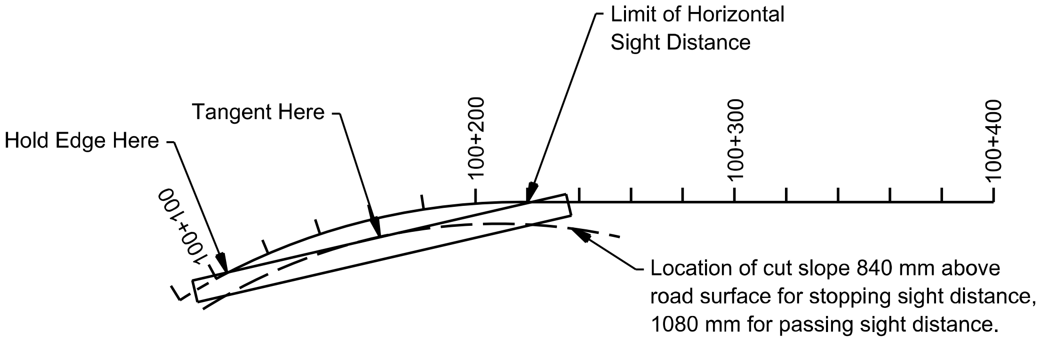


where: W_1 = *Major road travelled way width, m*
W_2 = *Adjusted width for skew, m*
Θ = *Intersection angle*

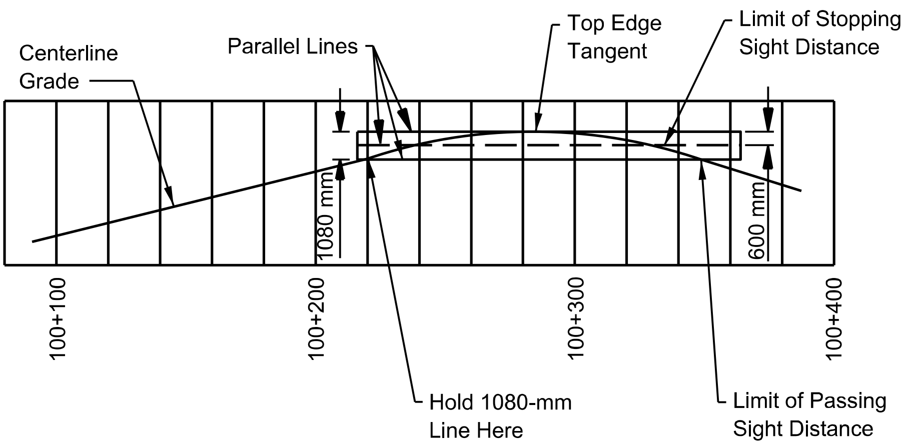
4.7 Measuring Sight Distance

By determining sight distances graphically on the plans at frequent intervals, the designer can review the overall layout and produce a more balanced design by making minor adjustments to the horizontal and vertical alignments. Figure 4-9 illustrates how to graphically measure sight distance in the plan and profile views. Because the view of the roadway may change rapidly in a short distance, it is desirable measure the sight distance for both direction of travel at each station. Measure both the horizontal and vertical sight distances; the shortest distance is the critical distance. In the case of a two-lane single carriageway, also measure the passing sight distance.

Figure 4-9: Measuring Sight Distances



(a) Plan



(b) Profile

5 HORIZONTAL ALIGNMENT

5.1 Overview

Chapter 5 presents criteria for the design of the elements of horizontal alignment. This includes horizontal curvature, superelevation, spiral curves, sight distance around horizontal curves, travelled way widening and general Controls for horizontal alignment.

5.2 Horizontal Curves

Horizontal curves are, in effect, transitions between two tangents. These deflection changes are necessary in virtually all roadway alignments to avoid impacts on a variety of field conditions (e.g. right of way, natural features, man-made features).

5.2.1 Types of Horizontal Curves

The following discusses the several types of horizontal curves that may be used to achieve the necessary roadway deflection:

1. Simple Curves. Simple curves are continuous arcs of constant radius that achieve the necessary roadway deflection without an entering or exiting taper. The radius (R) defines the circular arc that a simple curve will transcribe. All angles and distances for simple curves are computed in a horizontal plane. Figure 5-1 illustrates a simple horizontal curve. Because of their simplicity and ease of design, survey, and construction, the simple curve is typically used for horizontal curves on roadways.

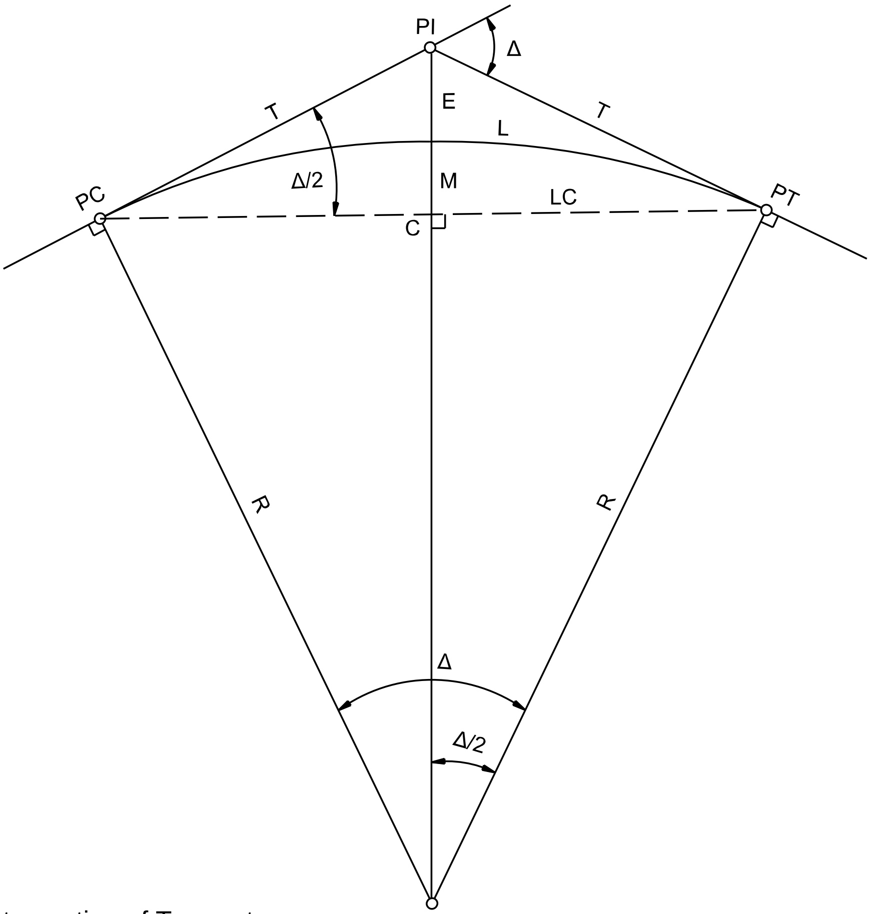
2. Compound Curves. Compound curves are a series of two or more simple curves with deflections in the same direction. Compound curves are generally only used on the roadway mainline to meet field conditions (e.g. to avoid obstructions that cannot be relocated) where a simple curve is not an option. Figure 5-2 illustrates a three-centred compound curve. Where a compound curve is used on a roadway mainline, the radius of the flatter circular arc (R_1) should not be more than 50% greater than the radius of the sharper circular arc (R_2). In other words, $R_1 \leq 1.5\ R_2$.

3. Spiral Curves. Spiral curves provide an entering transition into a simple curve with a variable rate of curvature along its layout. As an option to a simple curve, a restricted horizontal alignment and high-speed conditions may be conducive to the introduction of a spiral curve. Section 5.4 discusses spiral curves in further detail.

4. Reverse Curves. Reverse curves are two simple curves with deflections in opposite directions that are joined by a relatively short tangent distance. In rural areas, a minimum of 150 m should be provided between the PT and PC of the two curves for appearance. In urban areas, this distance may be 0 m (i.e. the PT of the first curve is the PC of the second curve). Section 5.3.7 further discusses reverse curves.

5. Broken-Back Curves. Broken-back curves are closely spaced horizontal curves with deflection angles in the same direction with an intervening, short tangent section (less than

Figure 5-1: Simple Curve Nomenclature



Notes:

PI = Point of Intersection of Tangents

PC = Point of Curvature (Beginning of Curve)

PT = Point of Tangency (End of Curve)

R = Radius of Curve, m

C = Mid-point of Long Chord

Δ = Deflection Angle Between Tangents or Central Angle, degrees

T = Tangent, Distance, m

LC = Length of Long Chord, m

L = Length of Curve, m

E = External Distance, m

$$E = T\tan\left(\frac{\Delta}{4}\right)$$

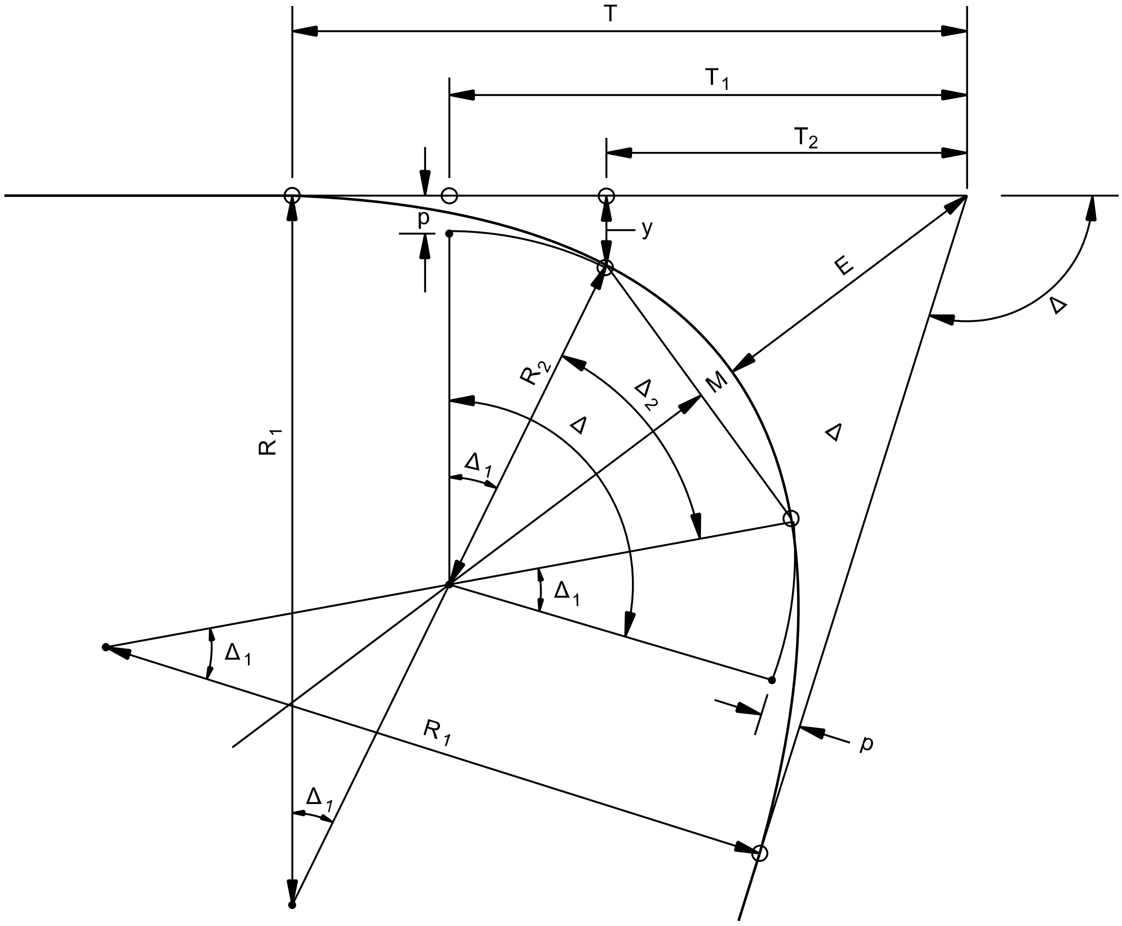
$$T = R\tan\left(\frac{\Delta}{2}\right)$$ where Δ is expressed as a decimal

$$LC = 2T\left(\cos\left(\frac{\Delta}{2}\right)\right)$$

$$M = R\left(1 - \cos\left(\frac{\Delta}{2}\right)\right)$$

$$L = \frac{\Delta R}{57.2958}$$

Figure 5-2: Three-Centred Compound Curve



Given: R_1, R_2, Δ_1, and p

$$T_1 = (R_2 + p)\tan\frac{\Delta}{2}$$

$$\Delta_1 = \cos^{-1}\left[\frac{R_1 - R_2 - p}{R_1 - R_2}\right]$$

$$T = T_1 + (R_1 - R_2)\sin\Delta_1$$

$$T_2 = T_1 - R_2\sin\Delta_1$$

$$E = \frac{R_2 + p}{\cos(\Delta/2)} - R_2$$

$$M = R_2 - (R_2\cos(\Delta/2 - \Delta_1))$$

$$y = (R_2 + p) - R_2\cos\Delta_1$$

$$\Delta_2 = \Delta - 2\Delta_1$$

$$\Delta = 2\Delta_1 + \Delta_2$$

Note: "p" is the offset location between the interior curve (extended) to a point where it becomes parallel with the tangent line.

500 m from PT to PC). Avoid broken-back curves on major roadways because of the potential for confusing a driver, problems with superelevation development, and the unpleasant view of the roadway that is created. Instead, use a single, flat simple curve or, if necessary, a compound curve.

5.2.2 General Theory

This section summarises the theoretical basis for the design of horizontal alignment.

5.2.2.1 Basic Equation

The point-mass formula is used to define vehicular operation around a curve. Where the curve is expressed using its radius, the basic equation for a simple curve is:

$$\mathbf{R_{min}} = \frac{\mathbf{V^2}}{\mathbf{127\,(e+f)}}$$

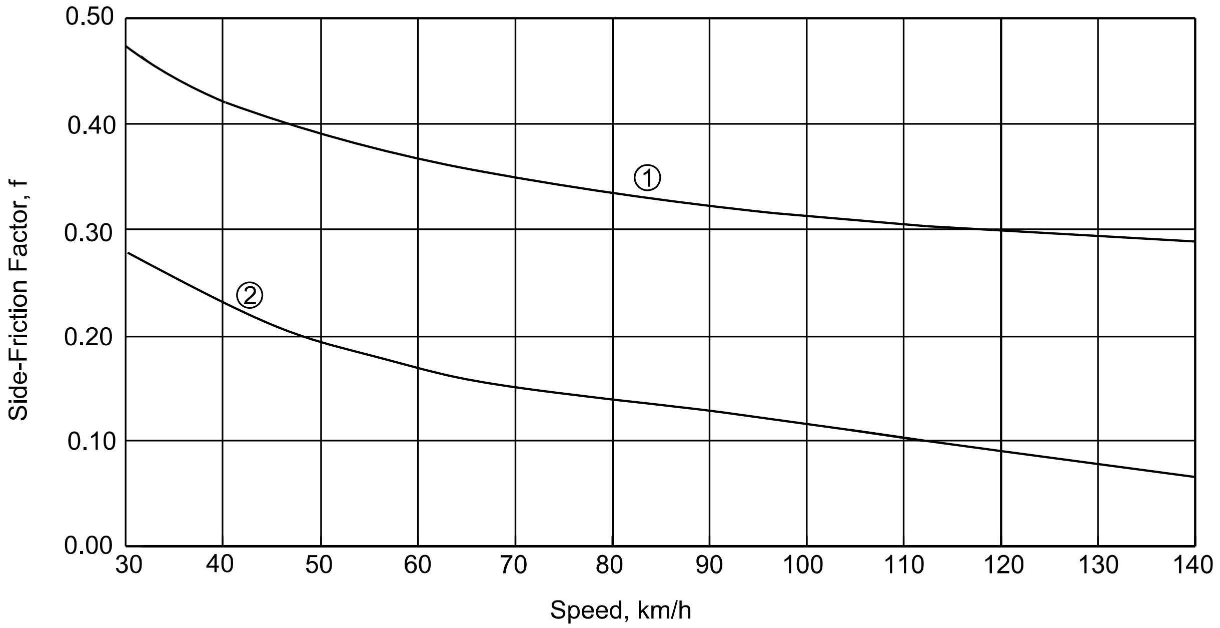
Equation 5.1: Basic Curve Equation

where: R_{min} = radius of curve, m
e = superelevation rate, decimal
f = side-friction factor, decimal
V = design speed, km/h

5.2.2.2 Side Friction Factors

AASHTO has established limiting side-friction factors (f) for various design speeds. It is important to realise that the f values used in design represent a threshold of driver discomfort and not the point of impending skid; see Figure 5-3.

Figure 5-3: Comparison of Side-Friction Factors



① *Estimated point of impending skid assuming smooth tires and wet PCC pavement.*

② *Side-friction factors used for design.* *Source:* (2)

5.2.2.3 Superelevation

Superelevation allows a driver to negotiate a curve at a higher speed than would otherwise be comfortable. Superelevation and side friction work together to offset the outward pull of the vehicle as it traverses the horizontal curve. In road design, it is necessary to establish limiting values of superelevation (e_{max}) based on the operational characteristics of the facility. See Section 5.3.1 for guidance on selecting e_{max}.

5.2.2.4 Distribution of Superelevation and Side Friction

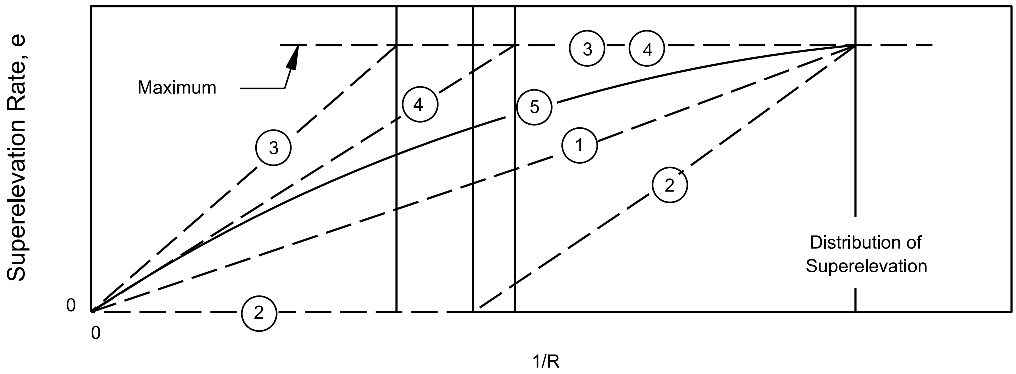
The minimum radius is based on selecting an e_{max} and f_{max} that apply to the facility. For curvature flatter than the minimum, a methodology must be applied to distribute superelevation and side friction for a given radius and design speed.

There are five methods for sustaining lateral acceleration on curves for a given design speed. These methods use e or f, or both; and are discussed below:

1. Method 1. Superelevation and side friction are directly proportional to the inverse of the radius (i.e. a straight-line exists between $1/R = 0$ and $1/R = 1/R_{min}$).
2. Method 2. Side friction is such that a vehicle travelling at the design speed has all lateral acceleration sustained by side friction on curves up to those designed for f_{max}. For sharper curves, f remains equal to f_{max} and superelevation is then used to sustain lateral acceleration until e reaches e_{max}. In this method, first f and then e are increased in inverse proportion to the radius of curvature.
3. Method 3. Superelevation is such that a vehicle travelling at the design speed has all lateral acceleration sustained by superelevation on curves up to those designed for e_{max}. For sharper curves, e remains at e_{max} and side friction is then used to sustain lateral acceleration until f reaches f_{max}. In this method, first e and then f are increased in inverse proportion to the radius of curvature.
4. Method 4. This method is the same as Method 3, except that it is based on average running speed instead of design speed.
5. Method 5. This method yields superelevation rates for which the superelevation counteracts nearly all centrifugal force at the average running speed and, therefore, considerable side friction is available for those drivers who are travelling near or above the design speed. Superelevation and side friction are in a curvilinear relation with the inverse of the radius of the curve, with values between those of Methods 1 and 3.

Figure 5-4 compares the relationship between superelevation and the inverse of the radius of the curve for these five methods.

Figure 5-4: Method of Distributing Superelevation and Side Friction



Legend: ◯ Method of distributing e and f, see Section 5.2.2.4

Source: (1)

The following two distribution methods are used for roadway design:

1. Open-Roadway Conditions. Superelevation and side friction are distributed using AASHTO Method 5. Section 5.3.1 presents the superelevation rates that result from the use of Method 5.

2. Low-Speed Urban Streets. For these conditions, superelevation and side friction are distributed using AASHTO Method 2. The practical effect of AASHTO Method 2 is that superelevation is rarely warranted on low-speed urban streets (V ≤ 70 km/h). Chapter 9 “Urban Streets” presents the superelevation rates that result from the use of Method 2. For this method of distribution, the superelevation rates may be calculated directly from Equation 5.1 using $f = f_{max}$.

5.2.3 Minimum Radii

Minimum radii are calculated directly from Equation 5.1 using the applicable values of e_{max} and f_{max}. In most cases, the road designer should limit the use of minimum radii because this results in the use of maximum superelevation rates. Table 5-1 (e_{max} = 8.0%), Table 5-2 (e_{max}=6.0%) and Table 5-3 (e_{max} = 4.0%) present the minimum radii for open-roadway conditions based on the design speed.

5.2.4 Maximum Deflection without Curve

It may be appropriate to omit a horizontal curve where very small deflection angles are present. As a guide, the road designer may retain deflection angles on the roadway mainline of 1 degree or less on low-speed urban roadways and 0 degree 30 minutes or less for open roadways. At these angles of deflection, the absence of a horizontal curve should not affect aesthetics.

Table 5-1: Minimum Radii (e_{max} = 8.0%)

Design Speed, V (km/h)	f_{max}	Minimum Radii, R_{min} (m)
30	0.28	20
40	0.23	41
50	0.19	73
60	0.17	113
70	0.15	168
80	0.14	229
90	0.13	304
100	0.12	394
110	0.11	502
120	0.09	667
130	0.08	832
140*	0.07	1029

Table 5-2: Minimum Radii (e_{max} = 6.0%)

Design Speed, V (km/h)	f_{max}	Minimum Radii, R_{min} (m)
30	0.28	21
40	0.23	43
50	0.19	79
60	0.17	123
70	0.15	184
80	0.14	252
90	0.13	336
100	0.12	438
110	0.11	561
120	0.09	756
130	0.08	951
140*	0.07	1188

Table 5-3: Minimum Radii (e_{max} = 4.0%)

Design Speed, V (km/h)	f_{max}	Minimum Radii, R_{min} (m)
30	0.28	22
40	0.23	47
50	0.19	86
60	0.17	135
70	0.15	203
80	0.14	280
90	0.13	375
100	0.12	492
110	0.11	635
120	0.09	872
130	0.08	1109
140*	0.07	1403

**Estimated by AD DMT.* *Source:* (1)

5.2.5 Minimum Length of Curve

For small deflection angles, horizontal curves should be sufficiently long to avoid the appearance of a kink. For aesthetics, a minimum 150 length of curve for a 5-degree central angle will eliminate the sense of abruptness, and the minimum length should be increased 30 m for each 1-degree decrease in the central angle. The minimum length for horizontal curves on main roadways should be three times the design speed ($L_{c,min} = 3V$). On high-speed controlled-access facilities that use flat curvature for aesthetic reasons, the desirable minimum length for curves should be double the minimum length described above, or $L_{c,des} = 6V$. Where the central angle is less than 5 degrees, Table 5-4 provides guidance for selecting minimum length of curve.

For central deflection angles greater than 5 degrees, use Equation 5.2 to calculate the minimum length of curve.

$$L = \frac{2\pi R\Delta}{360}$$

Equation 5.2: Formula for Angles more than 5 Degrees.

where: L = length of curve, m
Δ = deflection angle, degrees
R = radius of curve, m

Table 5-4: Minimum Lengths of Curve

Deflection Angle	Minimum Length (m)
5°00′	150
4°30′	165
4°00′	180
3°30′	195
3°00′	210
2°30′	225
2°00′	240

5.3 Superelevation Development

This section presents criteria for superelevation development when using open-roadway conditions. Open-roadway conditions apply to all rural facilities and to urban facilities where the design speed (V) ≥ 80 km/h. These types of facilities generally exhibit relatively uniform traffic operations. Therefore, for superelevation development, the flexibility normally exists to design horizontal curves with the more conservative AASHTO Method 5. This will maximise driver comfort and safety.

5.3.1 Superelevation Rates

The selection of a maximum rate of superelevation (e_{max}) depends upon several factors. These include design speed, urban/rural location, type of existing or expected roadside development, type of traffic operations expected, and prevalent climatic conditions. Table 5-5 identifies the allowable e_{max} for open-roadway conditions in Abu Dhabi.

Table 5-5: Selection of e_{max} (Open-Roadway Conditions)

Type of Facility	e_{max}
Urban freeways/expressways	4% – 6%
Rural freeways/expressways	4% – 6%
Urban arterials	4% – 6%
Rural arterials	6% – 8%
Rural collectors and local roads	6% – 8%
Loop ramps (interchanges)	6%

Based on the selection of emax and the use of AASHTO Method 5 to distribute (e) and (f),

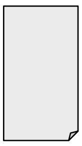
Table 5.6 (e_{max} = 8.0%), Table 5-7 (e_{max} = 6.0%) and Table 5-8 (e_{max} = 4.0%) allow the road designer to select the appropriate curve radius (R) for any combination of superelevation rate (e) and design speed (V). Note that the superelevation rates in the tables are expressed as percent. For the equations in which superelevation is included (e.g. superelevation runoff equation, point-mass equation for curve radius), e is expressed as a decimal.

5.3.2 Transition Lengths

The superelevation transition length is the distance required to transition the roadway from a normal crown section to the full design superelevation rate. The superelevation transition length is the sum of the tangent runout distance (L_t) and superelevation runoff length (L_r). See Figure 5-5.

5.3.2.1 Superelevation Runoff

Table 5-9 presents the superelevation runoff lengths (L_r) for 1+1 roadways (single carriageways) 2+2 roadways (dual carriageways) and for various combinations of superelevation rates and design speed. These lengths are calculated using Equation 5.3.



Equation 5.3: Superelevation Runoff

where:

where:	L_r	=	Superelevation runoff length, m
	w	=	Width of rotation (typically 3.65 m), m
	n_1	=	Number of lanes rotated
	e_d	=	Design superelevation rate
	Δ	=	Maximum relative gradient, % (see Table 5-10)
	b_w	=	Adjustment factor for number of lanes rotated (see Table 5-11)

Table 5-6: Superelevation Rate (e)
(e_{max} = 8.0%, AASHTO Method 5)

e (%)	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h
	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)
NC	184	443	784	1090	1490	1970	2440
RC	133	322	571	791	1090	1450	1790
2.2	119	288	512	711	976	1300	1620
2.4	107	261	463	644	885	1190	1470
2.6	97	237	421	587	808	1080	1350
2.8	88	216	385	539	742	992	1240
3.0	81	199	354	496	684	916	1150
3.2	74	183	326	458	633	849	1060
3.4	68	169	302	425	588	790	988
3.6	62	156	279	395	548	738	924
3.8	57	144	259	368	512	690	866
4.0	52	134	241	344	479	648	813
4.2	48	124	224	321	449	608	766
4.4	43	115	208	301	421	573	722
4.6	38	106	192	281	395	540	682
4.8	33	96	178	263	371	509	645
5.0	30	87	163	246	349	480	611
5.2	27	78	148	229	328	454	579
5.4	24	71	136	213	307	429	549
5.6	22	65	125	198	288	405	521
5.8	20	59	115	185	270	382	494
6.0	19	55	106	172	253	360	469
6.2	17	50	98	161	238	340	445
6.4	16	46	91	151	224	322	422
6.6	15	43	85	141	210	304	400
6.8	14	40	79	132	198	287	379
7.0	13	37	73	123	185	270	358
7.2	12	34	68	115	174	254	338
7.4	11	31	62	107	162	237	318
7.6	10	29	57	99	150	221	296
7.8	9	26	52	90	137	202	273
8.0	7	20	41	73	113	168	229

Table 5.6: Superelevation Rate (e)
(*Continued*)
(e_{max} = 8.0%, AASHTO Method 5)

e (%)	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h	V_d = 140 km/h*
	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)
NC	2970	3630	4180	4900	5360	6270
RC	2190	2680	3090	3640	4000	4680
2.2	1980	2420	2790	3290	3620	4248
2.4	1800	2200	2550	3010	3310	3887
2.6	1650	2020	2340	2760	3050	3581
2.8	1520	1860	2160	2550	2830	3319
3.0	1410	1730	2000	2370	2630	3092
3.2	1310	1610	1870	2220	2460	2894
3.4	1220	1500	1740	2080	2310	2718
3.6	1140	1410	1640	1950	2180	2562
3.8	1070	1320	1540	1840	2060	2423
4.0	1010	1240	1450	1740	1950	2298
4.2	948	1180	1380	1650	1850	2184
4.4	895	1110	1300	1570	1760	2081
4.6	847	1050	1240	1490	1680	1986
4.8	803	996	1180	1420	1610	1900
5.0	762	947	1120	1360	1540	1820
5.2	724	901	1070	1300	1480	1747
5.4	689	859	1020	1250	1420	1679
5.6	656	819	975	1200	1360	1616
5.8	625	781	933	1150	1310	1557
6.0	595	746	894	1100	1260	1502
6.2	567	713	857	1060	1220	1451
6.4	540	681	823	1020	1180	1402
6.6	514	651	789	982	1140	1357
6.8	489	620	757	948	1100	1314
7.0	464	591	724	914	1070	1274
7.2	440	561	691	879	1040	1236
7.4	415	531	657	842	998	1200
7.6	389	499	621	803	962	1165
7.8	359	462	579	757	919	1121
8.0	304	394	501	667	832	1029

**Estimated by DMT.*

Source: (1)

Table 5-7: Superelevation Rate (e)
(e_{max} = 6.0%, AASHTO Method 5)

e (%)	V_d = 20 km/h	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h
	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)
NC	194	421	738	1050	1440	1910	2360
RC	138	299	525	750	1030	1380	1710
2.2	122	265	465	668	919	1230	1530
2.4	109	236	415	599	825	1110	1380
2.6	97	212	372	540	746	1000	1260
2.8	87	190	334	488	676	910	1150
3.0	78	170	300	443	615	831	1050
3.2	70	152	269	402	561	761	959
3.4	61	133	239	364	511	697	882
3.6	51	113	206	329	465	640	813
3.8	42	96	177	294	422	586	749
4.0	36	82	155	261	380	535	690
4.2	31	72	136	234	343	488	635
4.4	27	63	121	210	311	446	584
4.6	24	56	108	190	283	408	538
4.8	21	50	97	172	258	374	496
5.0	19	45	88	156	235	343	457
5.2	17	40	79	142	214	315	421
5.4	15	36	71	128	195	287	386
5.6	13	32	63	115	176	260	351
5.8	11	28	56	102	156	232	315
6.0	8	21	43	79	123	184	252

e (%)	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h	V_d = 140 km/h*
	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)
NC	2880	3510	4060	4770	5240	6150
RC	2090	2560	2970	3510	3880	4680
2.2	1880	2300	2670	3160	3500	4120
2.4	1700	2080	2420	2870	3190	3758
2.6	1540	1890	2210	2630	2930	3452
2.8	1410	1730	2020	2420	2700	3189
3.0	1290	1590	1870	2240	2510	2961
3.2	1190	1470	1730	2080	2330	2761
3.4	1100	1360	1600	1940	2180	2585
3.6	1020	1260	1490	1810	2050	2428
3.8	939	1170	1390	1700	1930	2288
4.0	870	1090	1300	1590	1820	2161
4.2	806	1010	1220	1500	1720	2046
4.4	746	938	1140	1410	1630	1942
4.6	692	873	1070	1330	1540	1846
4.8	641	812	997	1260	1470	1759
5.0	594	755	933	1190	1400	1678
5.2	549	701	871	1120	1330	1603
5.4	506	648	810	1060	1260	1531
5.6	463	594	747	980	1190	1454
5.8	416	537	679	900	1110	1364
6.0	336	437	560	756	951	1189

*Estimated by DMT.

Source: (1)

Table 5-8: Superelevation Rate (e)
(e_{max} = 4.0%, AASHTO Method 5)

e (%)	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h	V_d = 140 km/h*
	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)	R(m)
NC	2170	2640	3250	3770	4470	4940	5820
RC	1490	1830	2260	2650	3180	3560	4210
2.2	1290	1590	1980	2331	2816	3173	3768
2.4	1110	1390	1730	2061	2512	2851	3398
2.6	944	1200	1510	1824	2250	2577	3083
2.8	802	1030	1320	1609	2020	2338	2810
3.0	690	893	1150	1419	1813	2128	2571
3.2	597	779	1010	1256	1628	1940	2360
3.4	518	680	879	1110	1458	1763	2162
3.6	448	591	767	976	1298	1591	1967
3.8	381	505	658	844	1136	1411	1760
4.0	280	375	492	637	874	1111	1406

*Estimated by DMT.

Source: (1)

Figure 5-5: Superelevation Transition Length

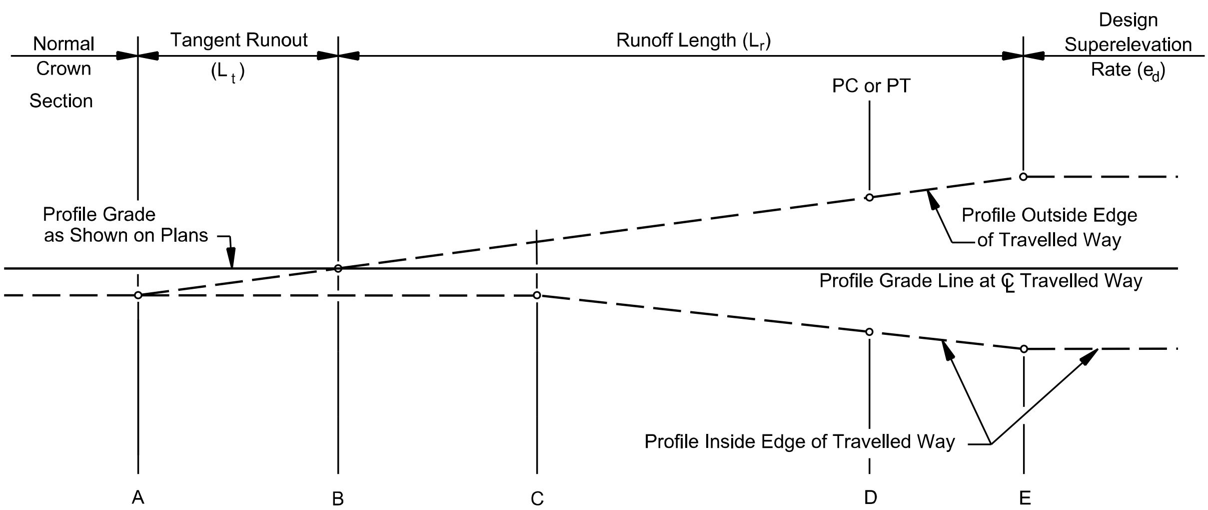


Table 5-9: Superelevation Runoff (L_r) for Horizontal Curves

e (%)	V_d = 20 km/h		V_d = 30 km/h		V_d = 40 km/h		V_d = 50 km/h		V_d = 60 km/h		V_d = 70 km/h		V_d = 80 km/h	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane roadway, 2 lanes rotated is typical for a 4-lane roadway, etc.													
	1	2	1	2	1	2	1	2	1	2	1	2	1	2
	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)	L_r (m)
2.0	9	14	10	14	10	15	11	17	12	18	13	20	14	22
2.2	10	15	11	16	11	17	12	18	13	20	14	22	16	24
2.4	11	16	12	17	12	19	13	20	14	22	16	24	17	26
2.6	12	18	12	19	13	20	14	22	16	23	17	26	19	28
2.8	13	19	13	20	14	22	16	23	17	25	18	27	20	30
3.0	14	20	14	22	15	23	17	25	18	27	20	29	22	32
3.2	14	22	15	23	16	25	18	27	19	29	21	31	23	35
3.4	15	23	16	24	17	26	19	28	20	31	22	33	24	37
3.6	16	24	17	26	19	28	20	30	22	32	24	35	26	39
3.8	17	26	18	27	20	29	21	32	23	34	25	37	27	41
4.0	18	27	19	29	21	31	22	33	24	36	26	39	29	43
4.2	19	28	20	30	22	32	23	35	25	38	27	41	30	45
4.4	20	30	21	32	23	34	24	37	26	40	29	43	32	48
4.6	21	31	22	33	24	35	25	38	28	41	30	45	33	50
4.8	22	32	23	35	25	37	27	40	29	43	31	47	35	52
5.0	23	34	24	36	26	39	28	42	30	45	33	49	36	54
5.2	23	35	25	37	27	40	29	43	31	47	34	51	37	56
5.4	24	36	26	39	28	42	30	45	32	49	35	53	39	58
5.6	25	38	27	40	29	43	31	47	34	50	37	55	40	60
5.8	26	39	28	42	30	45	32	48	35	52	38	57	42	63
6.0	27	41	29	43	31	46	33	50	36	54	39	59	43	65
6.2	28	42	30	45	32	48	34	52	37	56	41	61	45	67
6.4	29	43	31	46	33	49	35	53	38	58	42	63	46	69
6.6	30	45	32	48	34	51	37	55	40	59	43	65	48	71
6.8	31	46	33	49	35	52	38	56	41	61	45	67	49	73
7.0	31	47	34	50	36	54	39	58	42	63	46	69	50	76
7.2	32	49	35	52	37	56	40	60	43	65	47	71	52	78
7.4	33	50	36	53	38	57	41	61	44	67	48	73	53	80
7.6	34	51	36	55	39	59	42	63	46	68	50	75	55	82
7.8	35	53	37	56	40	60	43	65	47	70	51	77	56	84
8.0	36	54	38	58	41	62	44	66	48	72	52	79	58	86

Source: (1)

Table 5.9: Superelevation Runoff (L_r) for Horizontal Curves
(*Continued*)

e (%)	V_d = 90 km/h		V_d = 100 km/h		V_d = 110 km/h		V_d = 120 km/h		V_d = 130 km/h		V_d = 140 km/h*	
	Number of Lanes Rotated. Note that 1 lane rotated is typical for a 2-lane roadway, 2 lanes rotated is typical for a 4-lane roadway, etc.											
	1	2	1	2	1	2	1	2	1	2	1	2
	L_r (m)	L_r	L_r (m)	L_r	L_r (m)	L_r	L_r (m)	L_r	L_r (m)	L_r	L_r (m)	L_r
2.0	15	23	16	25	18	26	19	28	21	31	23	68
2.2	17	25	18	27	19	29	21	31	23	34	25	75
2.4	18	28	20	29	21	32	23	34	25	37	28	82
2.6	20	30	21	32	23	34	25	37	27	40	30	89
2.8	21	32	23	34	25	37	27	40	29	43	32	96
3.0	23	34	25	37	26	40	28	43	31	46	34	103
3.2	25	37	26	39	28	42	30	45	33	49	37	106
3.4	26	39	28	42	30	45	32	48	35	52	39	116
3.6	28	41	29	44	32	47	34	51	37	56	41	123
3.8	29	44	31	47	33	50	36	54	39	59	43	130
4.0	31	46	33	49	35	53	38	57	41	62	46	137
4.2	32	48	34	52	37	55	40	60	43	65	48	144
4.4	34	51	36	54	39	58	42	63	45	68	50	151
4.6	35	53	38	56	40	61	44	65	47	71	52	157
4.8	37	55	39	59	42	63	45	68	49	74	55	164
5.0	38	57	41	61	44	66	47	71	51	77	57	171
5.2	40	60	43	64	46	68	49	74	53	80	60	178
5.4	41	62	44	66	47	71	51	77	56	83	62	185
5.6	43	64	46	69	49	74	53	80	58	86	64	192
5.8	44	67	47	71	51	76	55	82	60	89	66	199
6.0	46	69	49	74	53	79	57	85	62	93	68	205
6.2	47	71	51	76	54	82	59	88	64	96	71	212
6.4	49	74	52	79	56	84	61	91	66	99	73	219
6.6	51	76	54	81	58	87	63	94	68	102	75	226
6.8	52	78	56	83	60	90	64	97	70	105	78	233
7.0	54	80	57	86	61	92	66	99	72	108	80	240
7.2	55	83	59	88	63	95	68	102	74	111	82	246
7.4	57	35	61	91	65	97	70	105	76	114	84	253
7.6	58	37	62	93	67	100	72	108	78	117	87	260
7.8	60	90	64	96	68	103	74	111	80	120	89	267
8.0	61	92	65	98	70	105	76	114	82	123	91	274

**Estimated by DMT.*

Source: (1)

Table 5-10: Maximum Relative Gradients

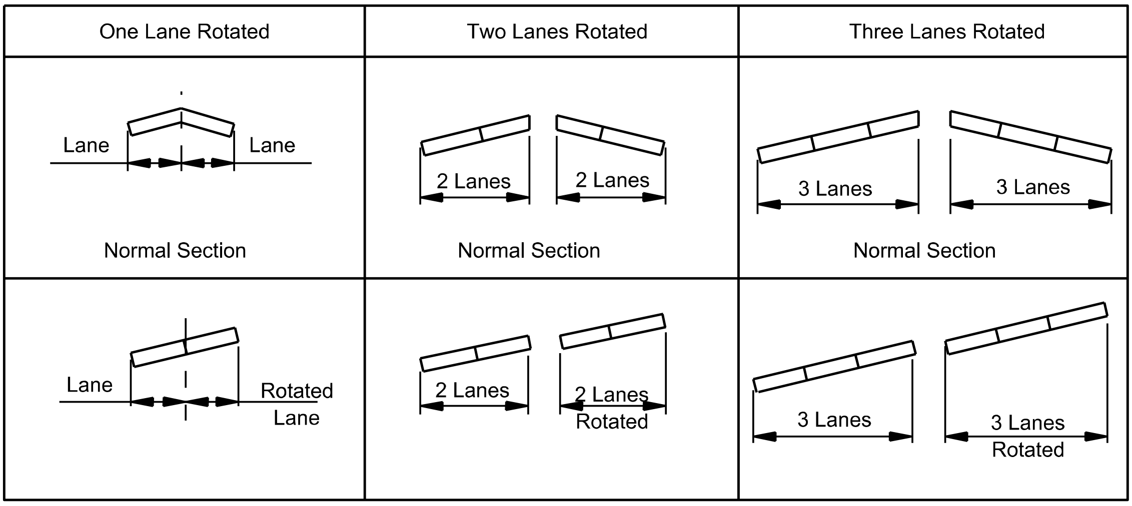
Design Speed (km/h)	Maximum Relative Gradient (%)	Equivalent Maximum Relative Slope
30	0.75	1:133
40	0.70	1:143
50	0.65	1:150
60	0.60	1:167
70	0.55	1:182
80	0.50	1:200
90	0.47	1:213
100	0.44	1:227
110	0.41	1:244
120	0.38	1:263
130	0.35	1:286
140*	0.32	1:313

Note: The maximum relative gradients are assumed to be measured between two lines set 3.65 m apart.
**Estimated by DMT.* *Source:* (1)

Table 5-11: Adjustment Factor for Number of Lanes Rotated

Number of Lanes Rotated (n_1)	Adjustment Factor* (b_w)	Length Increase Relative to One-Lane Rotated (n_1 x b_w)
1	1.00	1.00
1.5	0.83	1.25
2	0.75	1.50
2.5	0.70	1.75
3	0.67	2.00
3.5	0.64	2.25
4	0.63	2.50
4.5	0.61	2.75
5	0.60	3.00
5.5	0.59	3.25
6	0.58	3.50

*$b_w = (1 + 0.5(n_1-1))/n_1$



Source: (1)

5.3.2.2 Tangent Runout

The tangent runout distance is calculated as follows: The superelevation runoff lengths are calculated as follows:

$$L_t = \frac{e_{NC}}{e_d}\left(L_r\right)$$

Equation 5.4: Tangent Runout

where: L_t = Tangent runout distance, m
e_{NC} = Normal travel lane cross slope on tangent (typically, 0.02 m/m)

The relative longitudinal gradient for the tangent runout will be same as the relative longitudinal gradient for the superelevation runoff.

5.3.3 Application of Transition Length

Once the superelevation runoff (L_r) and tangent runout (L_t) have been calculated, the designer must determine how to fit the length into the horizontal and vertical planes. The following will apply:

1. Simple Curves. Typically, 67% of the superelevation runoff length will be placed on the tangent and 33% on the curve. Exceptions to this practice may be necessary to meet field conditions. The generally accepted range is 60% to 80% on the tangent and 40% to 20% on the curve. In extreme cases (e.g. to avoid placing any superelevation transition on a bridge or approach slab), the superelevation runoff may be distributed 50% to 100% on the tangent and 50% to 0% on the curve. This will usually occur only in urban areas with highly restricted right-of-way conditions. When considering the tangent runout distance, this results in a distribution of the total superelevation transition length of approximately 75% on the tangent and 25% on the curve. However, because the distribution of the superelevation transition length is not an exact science, the ratio should be rounded slightly (e.g. to the nearest 1.0-m increment) to simplify design and layout in construction.

2. Spiral Curves. The design superelevation runoff length (L_r) is typically assumed to fit the entire length of the spiral curve length (i.e. TS to SC and CS to ST). Therefore, all of the tangent runout is placed on the tangent before the TS and after the ST.

where:

TS = Tangent to spiral, common point of spiral and near transition
SC = Spiral to curve, common point of spiral and circular curve of near transition
CS = Curve to spiral, common point of circular curve and spiral of far transition
ST = Spiral to tangent, common point of spiral and tangent of far transition

3. Field Application (Vertical Profile). At the beginning and end of the superelevation transition length, angular breaks occur in the profile at the edge of the pavement if not smoothed. Field personnel usually smooth these abrupt angular breaks out during construction. This is usually accomplished by visually adjusting the wire used to control the vertical and horizontal position of the blacktop spreader or slip-form paver.

As a guide to eliminate angular breaks, the vertical curve transitions should have a length numerically equivalent to approximately 20% of design speed in km/h. In addition, designers should graphically or numerically investigate the transition areas to identify potential flat spots for drainage before finalizing construction plans.

4. Ultimate Development. If the proposed facility is planned for an ultimate development of additional lanes, the designer should, where practical, reflect this length in the initial superelevation transition application. For example, a 2+2 divided facility may be planned for an ultimate 3+3 divided facility. Therefore, the superelevation transition length for the initial 2+2 facility should be consistent with the future requirements of the 3+3 facility.

5.3.4 Axis of Rotation

The following define the typical location of axis or rotation for common road conditions:

5.3.4.1 Two-Lane Roadways (Single Carriageway)

The axis of rotation may be about the centreline of the roadway on two-lane, two-way roadways. This method will yield the least amount of elevation differential between the pavement edges and their normal profiles. Occasionally, it may be necessary to rotate the pavement about the inside or outside edge of the travelled way. This may be necessary to meet field conditions (e.g. drainage on a curbed facility, roadside development). Note that, in this case, two travel lane widths will be rotated and the superelevation runoff should be lengthened according to Table 5-11.

5.3.4.2 Multilane Divided Roadways (Dual Carriageways)

The axis of rotation will typically be about the two inside edge of travelled ways for a multilane divided facility with a concrete barrier, a raised median > 5.0 m or a depressed median ≥12 m. Where the median edges are used as the axes of rotation, the median will remain in the same horizontal plane throughout the curve.

Several features may significantly influence superelevation development for multilane divided roads. These could include guardrail, median barriers, drainage, and major at-grade intersections. If a major crossroad intersection is present where the median width is greater than 5.0 m, it is recommended that the entire cross section of the mainline be rotated about the centreline of the roadway. This method of rotation will provide better operations for cross road traffic at the intersection.

The designer should carefully consider the intended function of all road features and ensure that the superelevated section and selected axis of rotation does not compromise traffic operations. In addition, the designer should consider the likely ultimate development of the facility and select an axis of rotation that will lend itself to future expansion.

5.3.4.3 Multilane Roads with Narrow/Flush Medians

For divided roads with narrow raised (< 5.0 m) or flush medians, develop the superelevation by rotating the roadway about its centreline.

5.3.5 Shoulder Superelevation

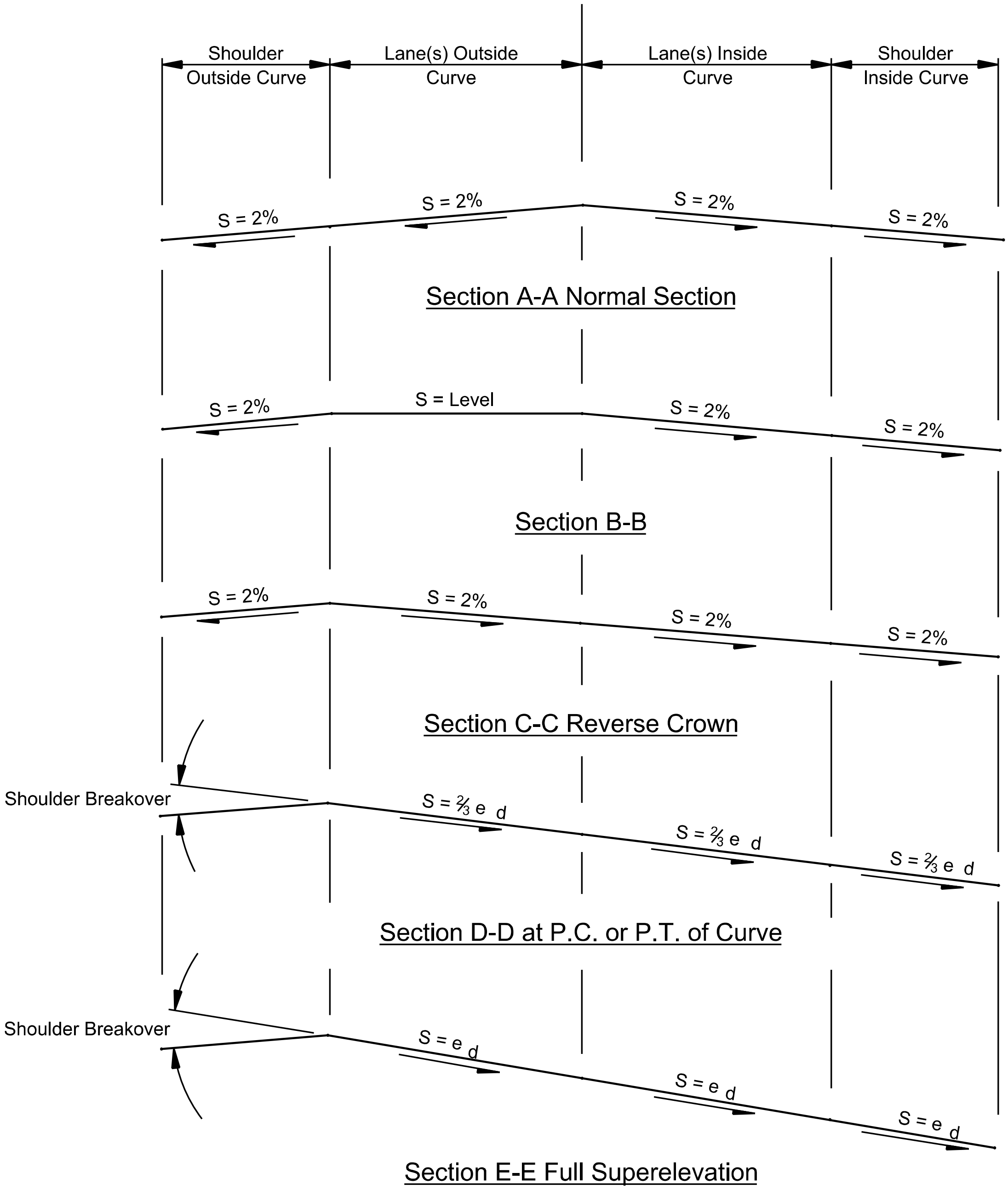
Figure 5-6 illustrates shoulder treatment on superelevated sections. The following discusses specific criteria for treatment of shoulders in superelevated sections.

5.3.5.1 High Side (Outside Shoulder)

On the high side of superelevated sections, there will be a break in the cross slopes of the travel lane and shoulder; see Figure 5-6. The following criteria will apply to this shoulder break over:

1. <u>Algebraic Difference</u>. The algebraic difference between the travelled way and shoulder should not exceed 8%.

Figure 5-6: Shoulder Treatment Through Superelevated Curve



Note: See Section 5.3.5 for criteria on treatment of shoulders through superelevated curves.

2. Shoulder Slope. Where the maximum design superelevation (e_d) is ≤ 6%, retain the normal tangent shoulder slope of 2% throughout the horizontal curve. Slope the shoulder away from the travelled way; see Figure 5-6.

Where e_d > 6%, the maximum algebraic difference criteria from Comment #1 would be exceed with a 2% shoulder. For example, if e_d = 7.5%, a 2% shoulder slope away from the travel lane would yield a 9.5% break over, which exceeds the maximum 8% break over. Where e_d > 6%, rotate the outside shoulder with the outside travel lanes so that they both remain in the same plane.

3. Shoulder as Deceleration Lane. At some intersections, drivers may use a paved shoulder as a right-turn lane on a superelevated horizontal curve. Chapter 10 “Intersections” presents cross slope break over criteria between a turning roadway and a through travel lane at an intersection at-grade. Where the shoulder is used by turning vehicles, the designer may want to use a maximum break over of 4% to 5% rather than the 8% maximum break over.

5.3.5.2 Low Side (Inside Shoulder)

On the low side of a superelevated section, the shoulder is superelevated concurrently with the travel lane until the design superelevation is reached (i.e. the inside shoulder and travel lane will remain in a plane section). See Figure 5-6.

5.3.6 Compound Curves

Superelevation development for compound curves requires the consideration of several factors. For two-lane roadways, these are discussed in the following sections for two cases:

- Case I: The distance between the PC and PCC is 100 m or less.
- Case II: The distance between the PC and PCC is greater than 100 m.

5.3.6.1 Case I

For Case I, superelevation development for compound curvature on two-lane roadways should meet the following objectives:

1. Relative Longitudinal Gradient (RS). A uniform RS should be provided throughout the superelevation transition (from normal section to full superelevation at the PCC).

2. Superelevation at PCC. The criteria in Section 5.3.1 (open-roadway conditions) yields the design superelevation (e_d) for the second curve. The design superelevation should be reached at the PCC.

3. Superelevation at PC. Section 5.3.1 yields the design superelevation (e_{d1}) for the first curve. At the PC, 2/3 e_{d1} should be reached.

4. Superelevation Runoff Length. Section 5.3.2.1 will yield the superelevation runoff (L_r) for the first curve. The superelevation should be developed such that 2/3 L_r is reached at the PC.

5. Tangent Runout. (TR_{ML}) will be determined as described in Section 5.3.2.2.

To meet all or most of these objectives, the designer may need to try several combinations of curve lengths, radii, and longitudinal gradients to find the most practical design.

5.3.6.2 Case II

For Case II, the distance between the PC and PCC (> 100 m) is normally large enough to allow the two curves to be evaluated individually. Therefore, the superelevation development on two-lane roadways should meet the following objectives for Case II:

1. First Curve. Superelevation should be developed assuming the curve is an independent simple curve. Therefore, the criteria in Sections 5.3.1 and 0 for superelevation rate, transition length, and distribution between tangent and curve apply.

2. Intermediate Treatment. Superelevation for the first curve (e_1) is reached a distance of 1/3 the superelevation runoff length beyond the PC. e_1 is maintained until it is necessary to develop the needed superelevation rate (e_2) for the second curve.

3. Second Curve. Assuming the second curve has a smaller radii than the first curve, a higher rate of superelevation will be required ($e_2 > e_1$). e_2 should be reached at the PCC. The distance needed for the additional superelevation development is not specified, except that the maximum RS for the roadway design speed must not be exceeded. One logical treatment would be to apply the same RS used for the superelevation transition of the first curve. This would provide a uniform change in gradient for the driver negotiating the compound curve.

5.3.6.3 Multilane Roadways

Superelevation development for compound curvature on multilane roadways should, as practical, be designed to:

- meet the principles of superelevation development for simple curves on multilane roadways (see applicable criteria in Section 5.3); and

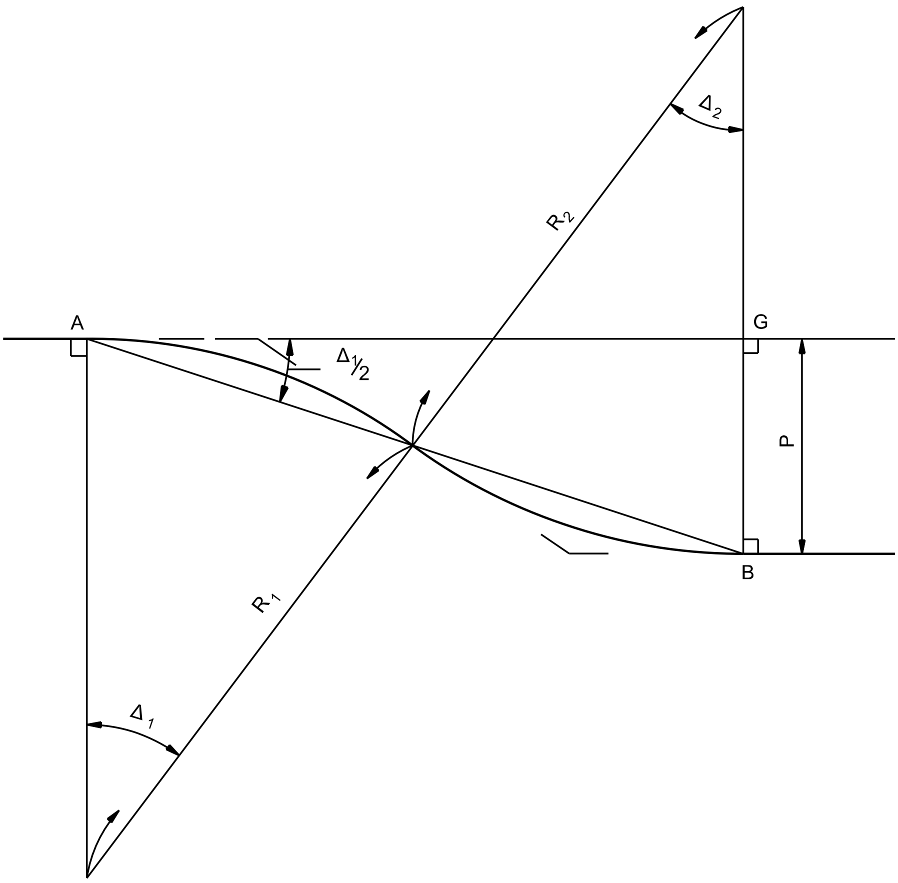
- meet the objectives for Case I or Case II as described for two-lane roadways.

The treatment for multilane roadways will be determined on a case-by-case basis, reflecting individual site conditions.

5.3.7 Reverse Curves

Reverse curves are two closely spaced simple curves with deflections in opposite directions. See Figure 5-7. For this situation, it may not be practical to achieve a normal crown section between the curves. A plane section continuously rotating about its axis can be maintained between the two curves, if they are close enough together. The designer should adhere to the applicable superelevation development criteria for each curve. The following will apply to reverse curves:

Figure 5-7: Reverse Curves to Parallel Tangents



EQUAL RADII	UNEQUAL RADII
Given: Radius and BG	Given: R_1, AG, and P
1. $R_1 = R_2$	1. $\Delta_1 = \Delta_2$
2. $\Delta_1 = \Delta_2$	2. $AB = \sqrt{AG^2 + P^2}$
3. $BG = P$	3. $R_2 = \frac{(AB)^2}{2P} - R_1$
4. $\cos \Delta_1 = \frac{R_1 - \frac{1}{2}P}{R_1}$	4. $\sin \Delta_1 = \frac{AG}{R_1 + R_2}$
5. $AG = \sqrt{4PR_1 - P^2}$	5. $\cos \Delta_1 = \frac{R_1 + R_2 - P}{R_1 + R_2}$
6. $\sin \Delta_1 = \frac{AG}{2R_1}$	6. $\tan \Delta_1 = \frac{AG}{R_1 + R_2 - P}$
7. $\tan \Delta_1 = \frac{AG}{2R_1 - P}$	

1. Normal Section. The designer should not attempt to achieve a normal tangent section between reverse curves unless the normal section can be maintained for a minimum of two seconds of travel time, and the superelevation transition requirements can be met for both curves. These criteria yield the following minimum tangent distance (between PT of first curve and PC of second curve):

$$L_{tan} = 0.67L_{r1} + L_{t1} + 2(0.278V) + L_{t2} + 0.67L_{r2}$$

Equation 5.4: Tangent Distance Between PT and PC

where: L_{tan} = Tangent distance between PT and PC, m
L_{r1} = Superelevation runoff length for first curve, m
L_{t1} = Tangent runout length for first curve, m
V = Design speed, km/h
L_{t2} = Tangent runout length for second curve, m
L_{r2} = Superelevation runoff length for second curve, m

As a modification to the equation for L_{tan}, developing a normal section is acceptable if between 60% to 80% of L_{r1} and L_{r2} can be provided on the intervening tangent.

2. Continuously Rotating Plane. If a normal section is not provided, the pavement will be continuously rotated in a plane about its axis. In this case, the minimum distance between the PT and PC will be that needed to meet the superelevation transition requirements:

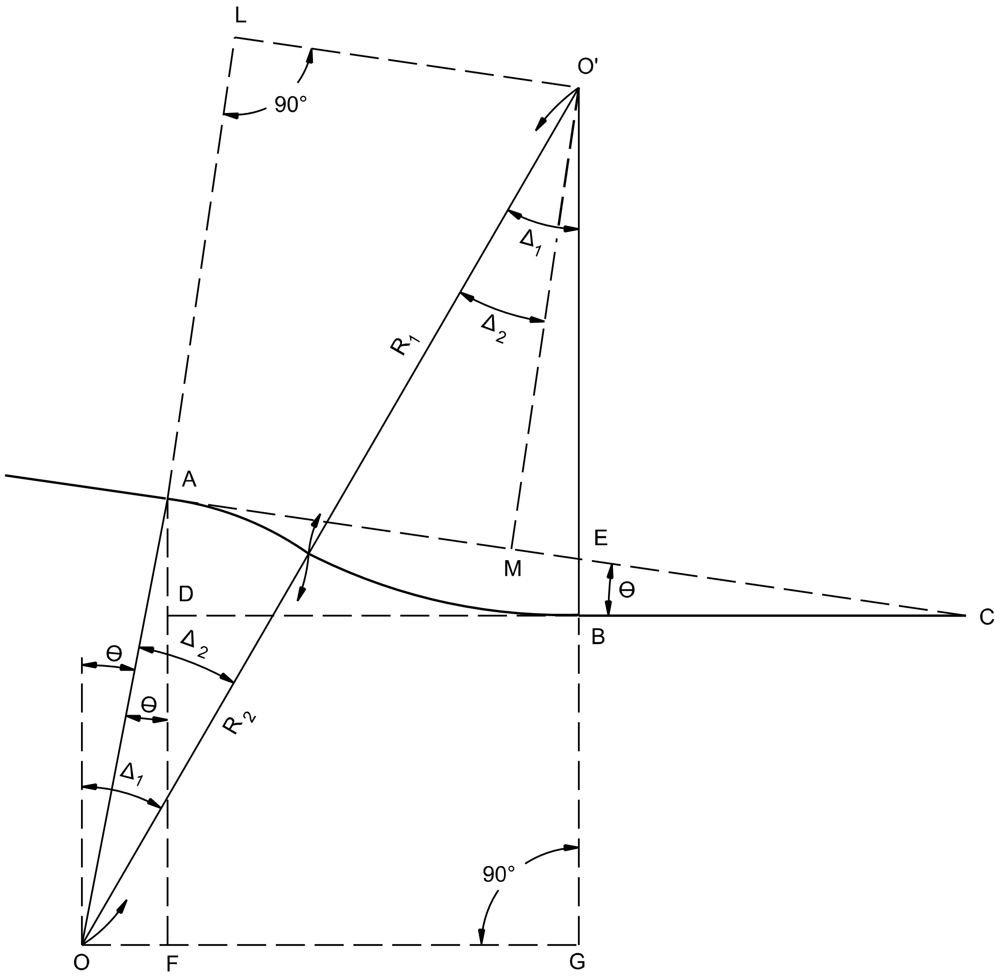
$$L_{tan} = 0.67L_{r1} + L_{t1} + L_{t2} + 0.67L_{r2}$$

Equation 5.5: Tangent Distance

Where terms are as defined in Comment #1. As a modification to the equation for Ltan, it is acceptable to provide between 60% to 80% of Lr1 and Lr2 on the intervening tangent.

3. Reverse Curve Details. Figure 5-7 and Figure 5-8 present mathematical details for various applications to the design of reverse curves. Figure 5-9 illustrates superelevation development for reverse curves.

Figure 5-8: Reverse Curves (Tangents not Parallel)



Given: θ, AD, R_1, and R_2

Required: Δ_1 and Δ_2

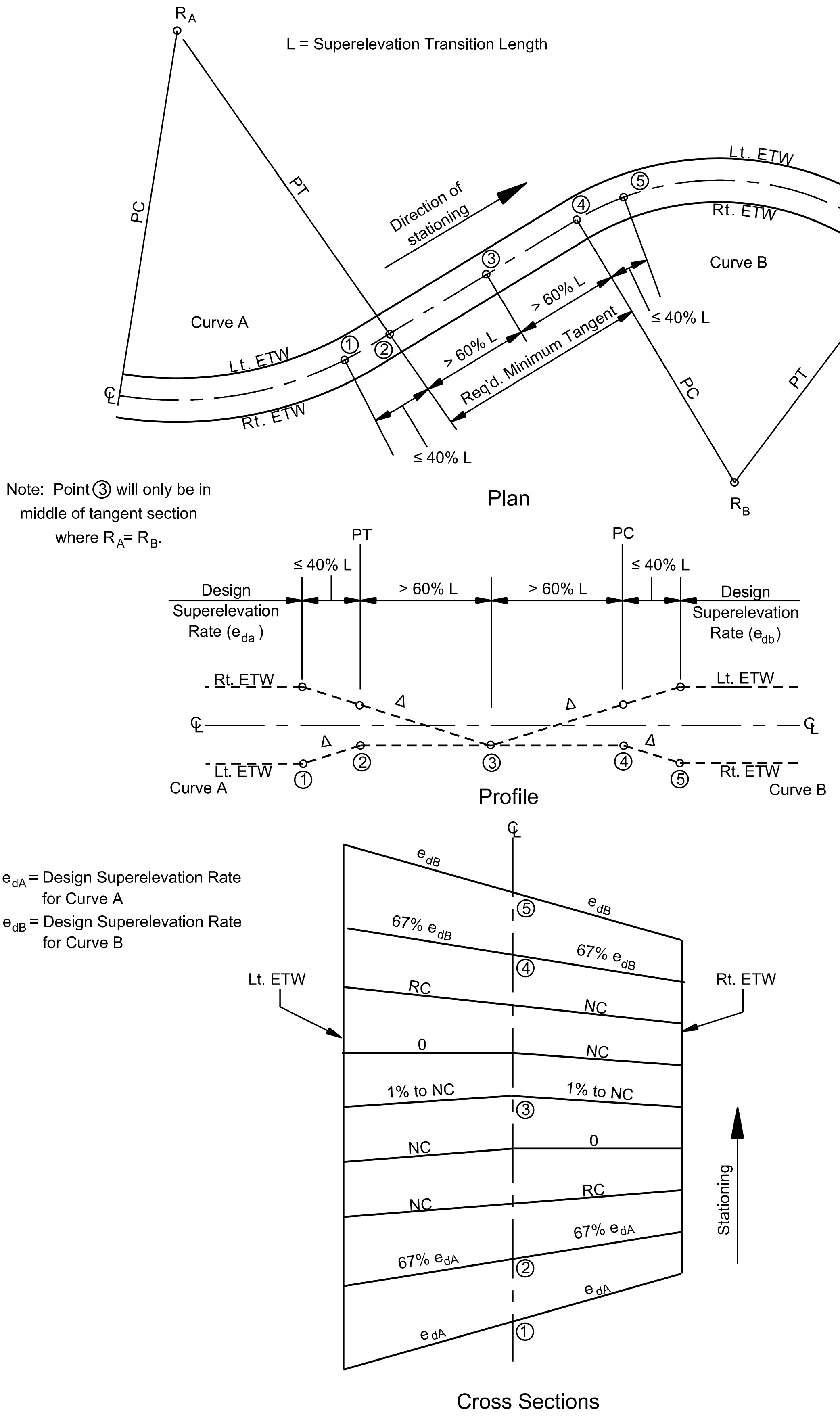
1. $AC = \dfrac{AD}{\sin\theta}$

2. $BG = DF = R_2 \cos\theta - AD$

3. $\cos\Delta_1 = \dfrac{R_1 + BG}{R_1 + R_2}$

4. $\Delta_2 = \Delta_1 - \theta$

Figure 5-9: Superelevation Development for Reverse Curves



5.3.8 Superelevation Development Figures

The following figures provide typical superelevation development diagrams for the most common design situations:

- Figure 5-10 illustrates a single carriageway rotated about the centreline.
- Figure 5-11 illustrates a single carriageway rotated about the inside of a curve.
- Figure 5-12 illustrates a single carriageway rotated about the outside of a curve.
- Figure 5-13 illustrates a dual carriageway with a depressed median rotated about the median edges.
- Figure 5-14 illustrates a dual carriageway with a raised median rotated about the median edges.
- Figure 5-15 illustrates superelevation development for compound curves.

5.3.9 Examples

The following examples illustrate the application of the superelevation development criteria in Section 5.3 to specific site conditions. In all of the examples, a negative cross slope or superelevation rate slopes down from left to right and a positive cross slope or superelevation rate slopes up from left to right.

Example 5.3-1 (Two-Lane Single Carriageway)

Given: Facility —Two-lane rural collector
Travel lane cross slope = 2% (on tangent) = 0.020 m/m = e_{NC}
Lane width = 3.65 m
Design Speed = 100 km/h
R = 750 m
PC = Station 20+000
PT = Station 20+500

Problem: With the axis of rotation about the pavement centreline, determine the applicable details for superelevation development for the horizontal curve, including:

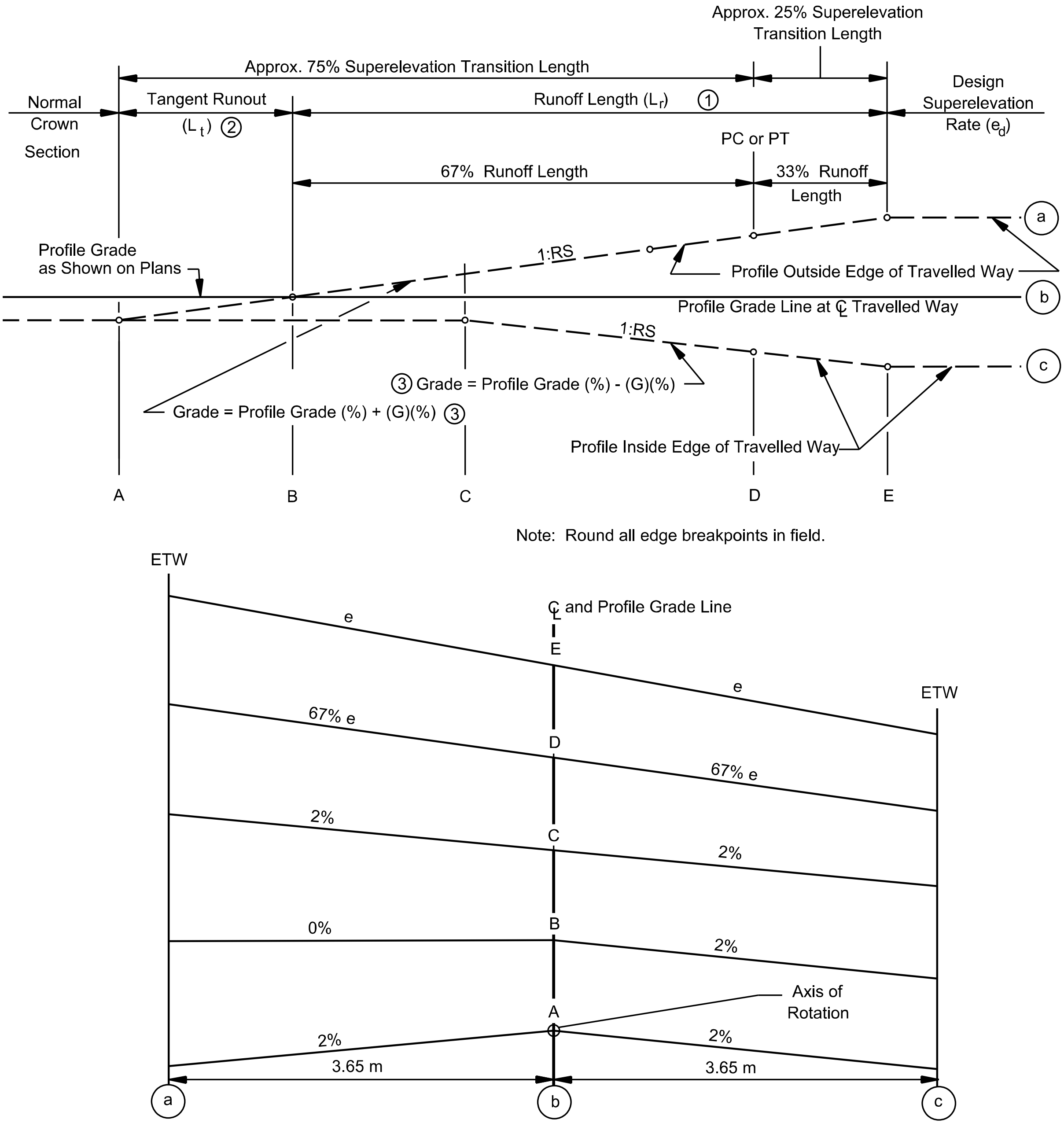
- e_{max}
- design superelevation rate, e_d
- superelevation runoff length, L_r
- tangent runout length, L_t, and
- relative longitudinal gradient for the two outside edges of travelled way.

Solution: The details of the superelevated curve are determined as follows, and Figure 5-16 presents the completed example and shows all stationing.

1. Determine e_{max}. As discussed in Section 5.3.1, e_{max} = 6% or 8% for rural collectors and open-roadway conditions. For this example, use e_{max} = 0.08.

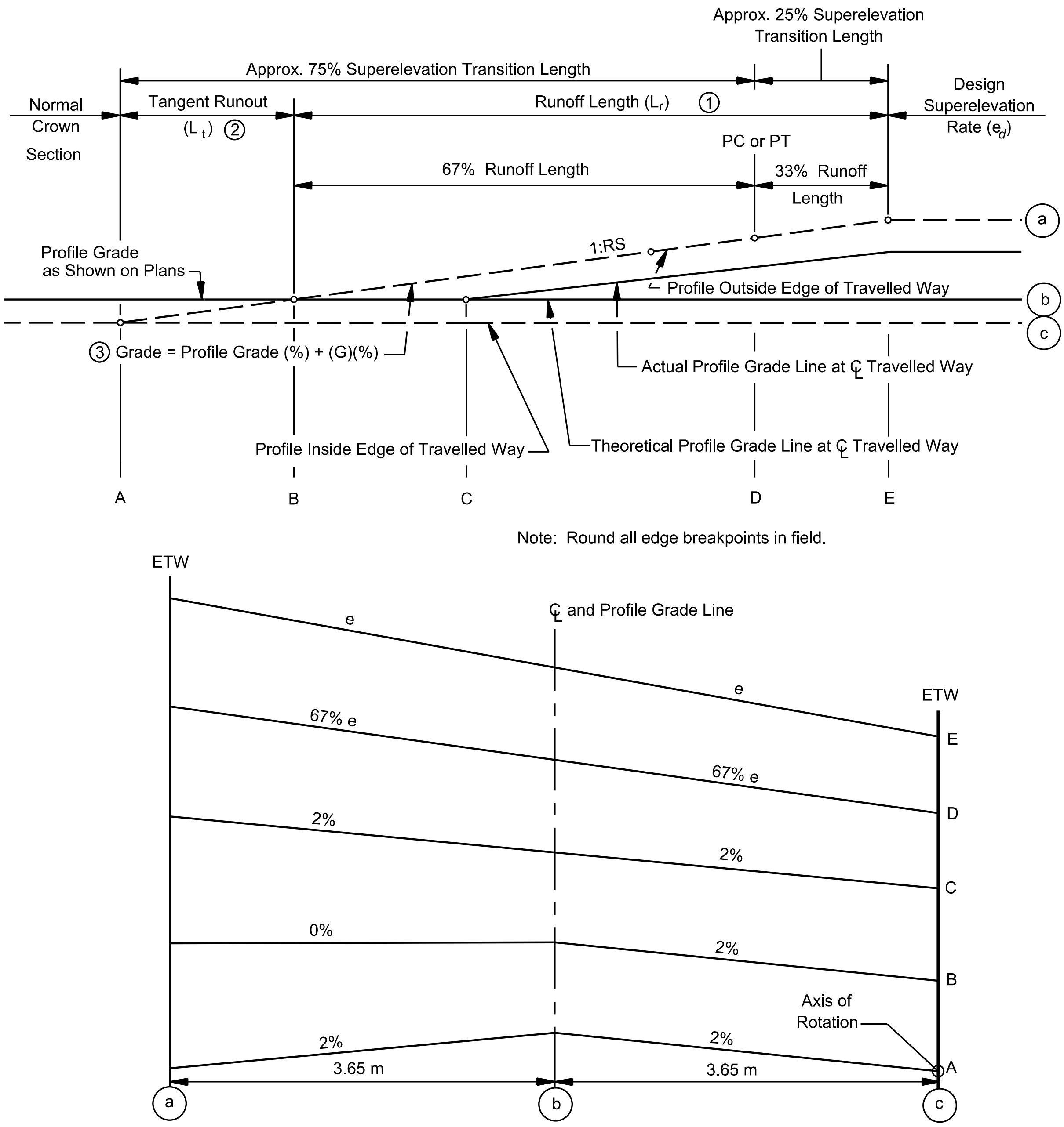
2. Determine Design Superelevation Rate (ed). From figure 5.6, ed = 0.060 m/m for radii between 746 m and 781 m and Vd = 100 km/h.

Figure 5-10: Axis of Rotation About Centreline (Single Carriageway)



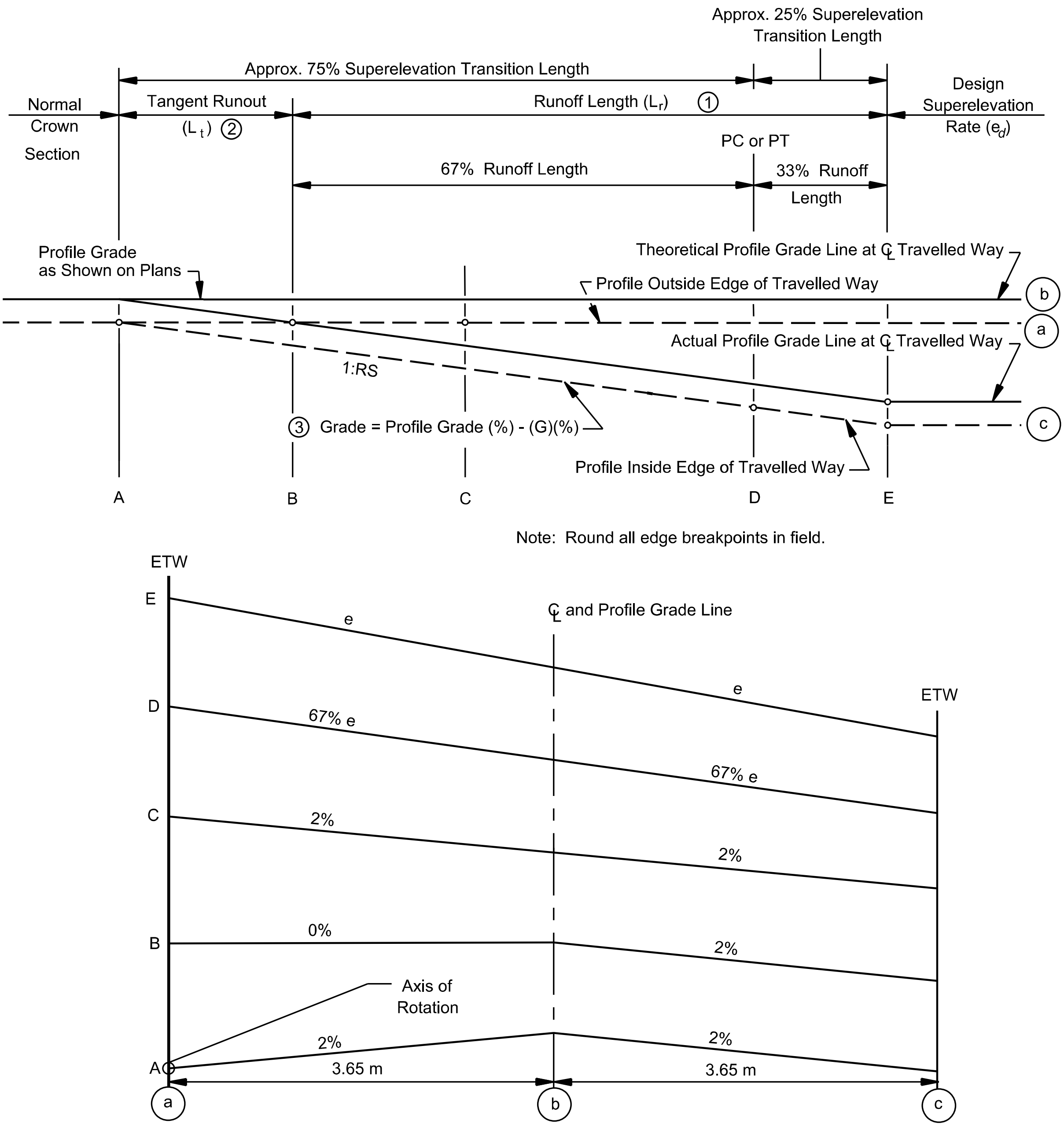
① See Section 5.3.2 for superelevation runoff calculations.
② See Section 5.3.2 for tangent runout calculations.
③ Grade difference G(%). See Table 5-10 for values of G.

Figure 5-11: Axis of Rotation About Inside Edge (Single Carriageway)



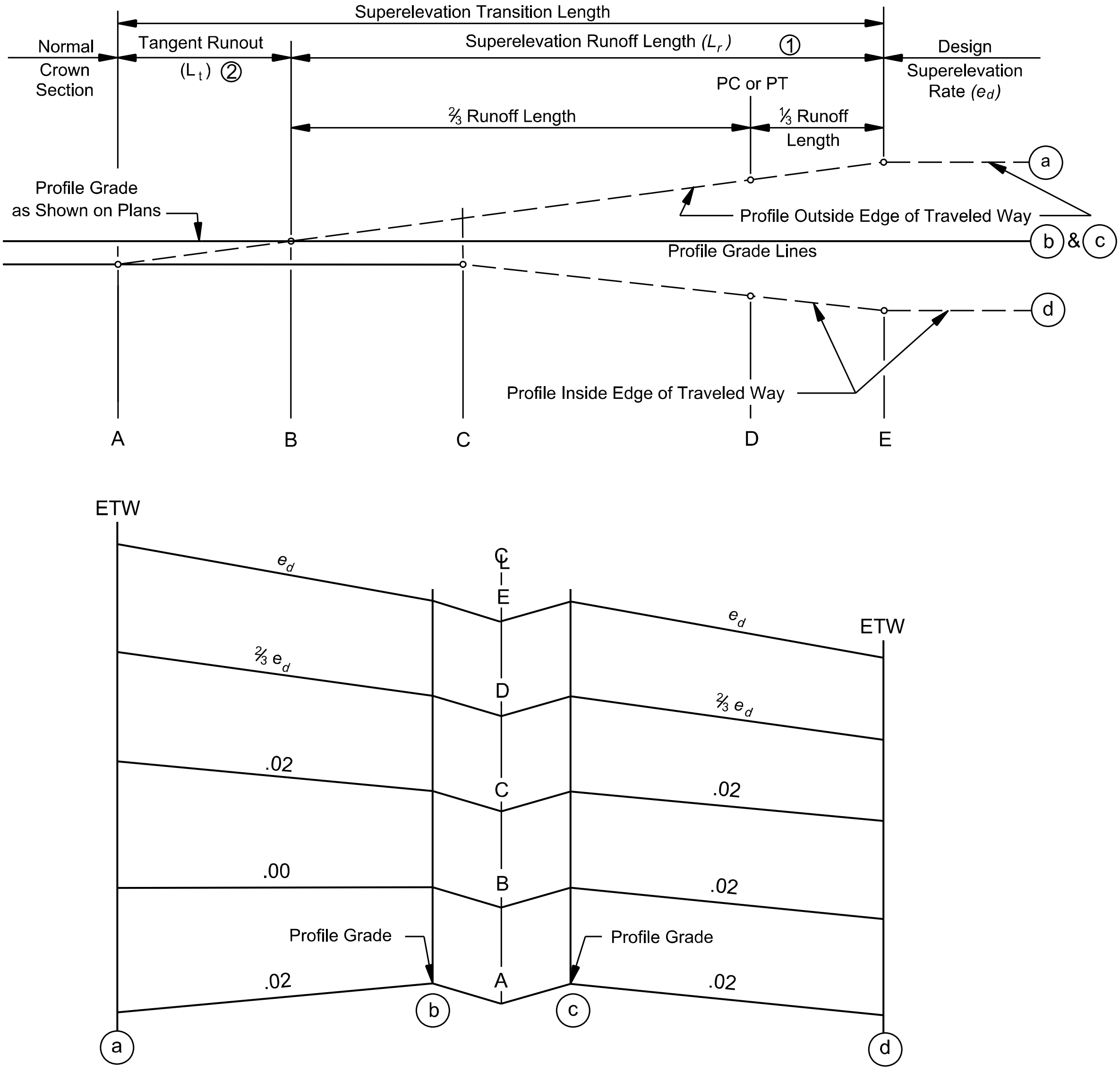
① See Section 5.3.2 for superelevation runoff calculations.
② See Section 5.3.2 for tangent runout calculations.
③ Grade difference G(%). See Table 5-10 for values of G.

Figure 5-12: Axis of Rotation About Outside Edge (Single Carriageway)



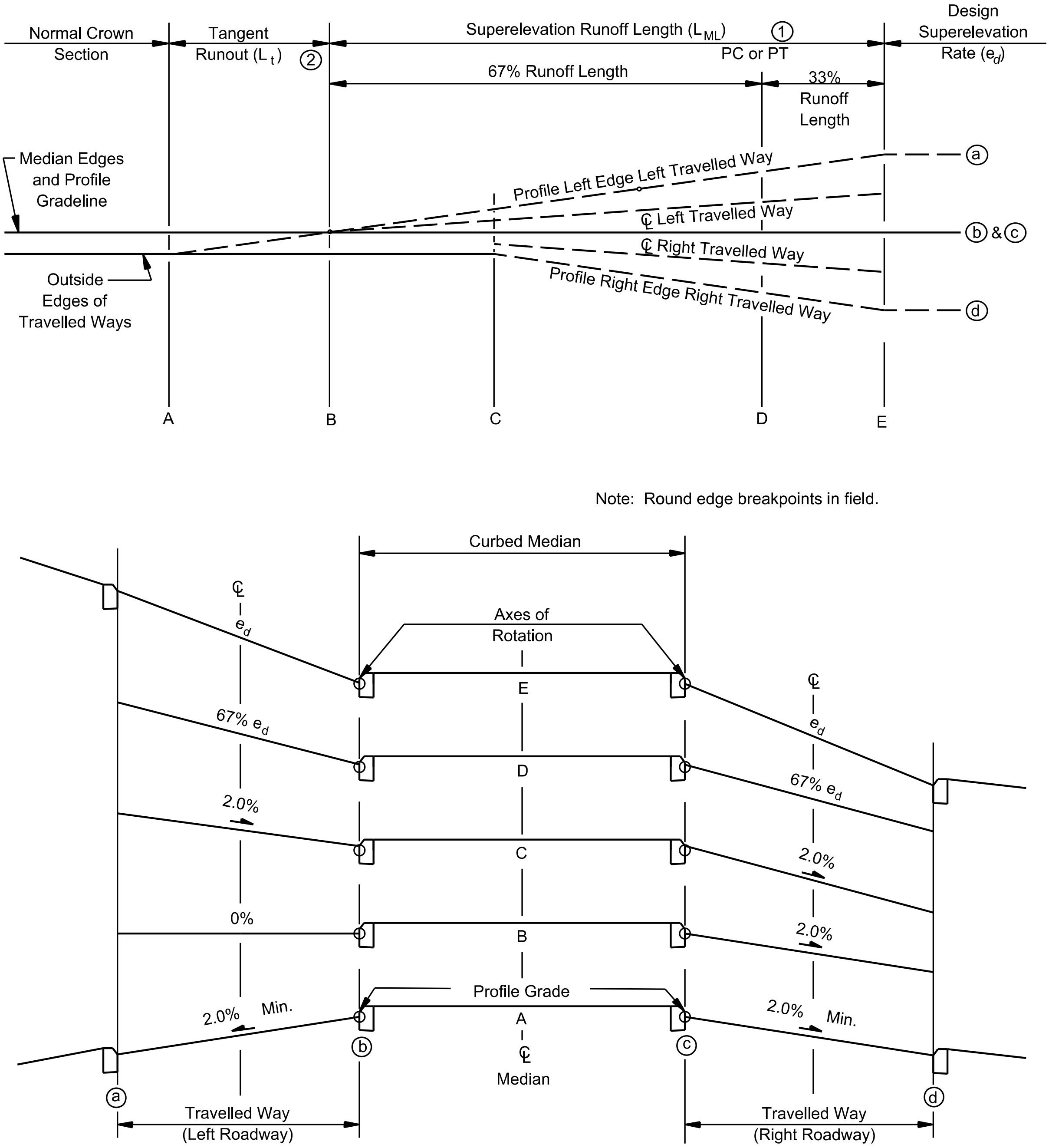
① See Section 5.3.2 for superelevation runoff calculations.
② See Section 5.3.2 for tangent runout calculations.
③ Grade difference G(%). See Table 5-10 for values of G.

Figure 5-13: Axis of Rotation for Dual Carriageway (Depressed Median)



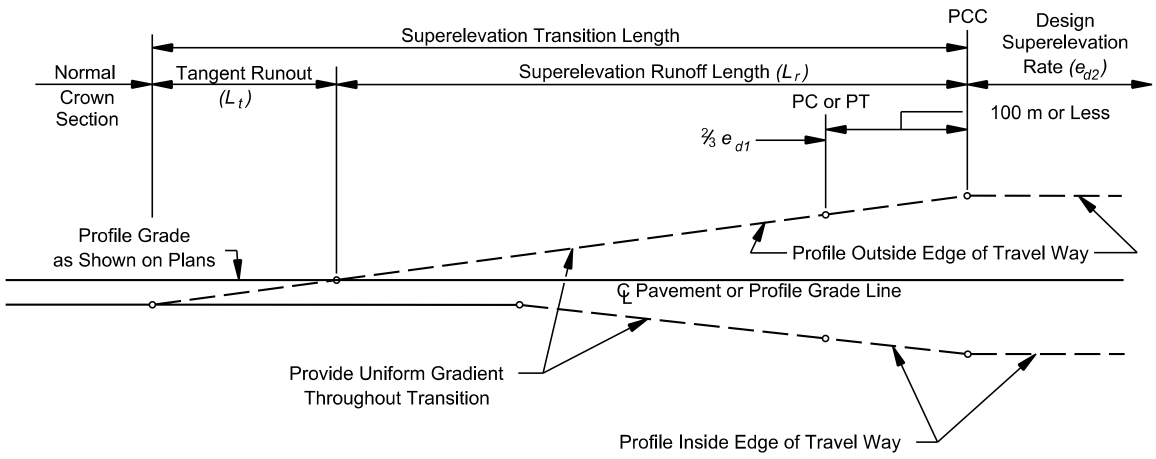
① See Section 5.3.2 for superelevation runoff calculations.
② See Section 5.3.2 for tangent runout calculations.
③ Grade difference G(%). See Table 5-10 for values of G.

Figure 5-14: Axis of Rotation for Dual Carriageway (Raised Median)

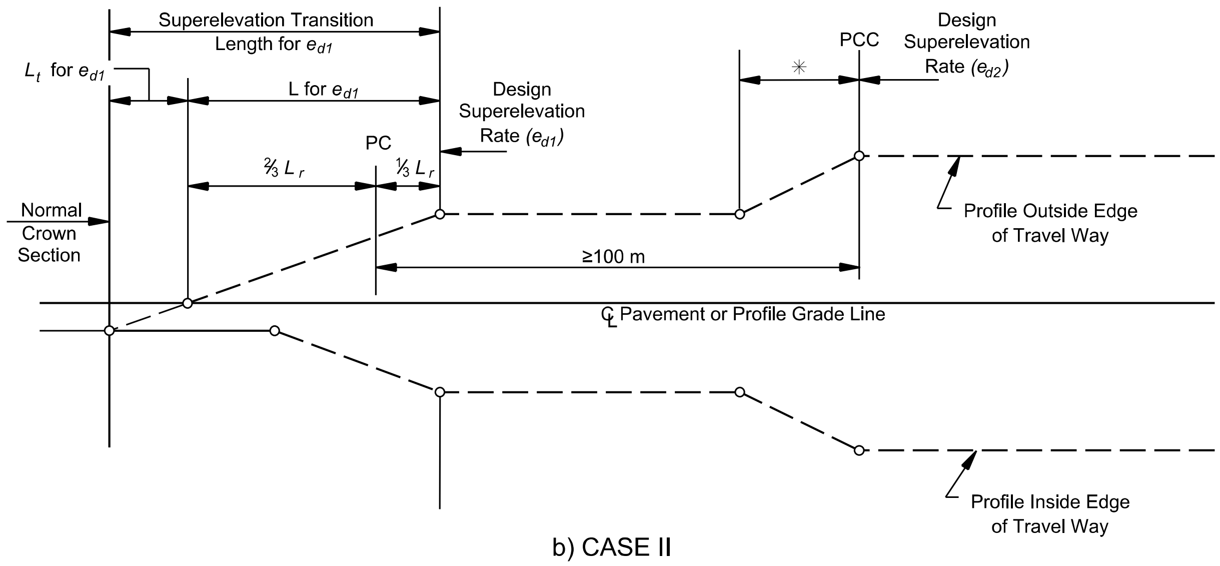


① See Section 5.3.2 for superelevation runoff calculations.
② See Section 5.3.2 for tangent runout calculations.
③ Grade difference G(%). See Table 5-10 for values of G.

Figure 5-15: Axis of Rotation About Centreline (Compound Curves)



a) CASE I

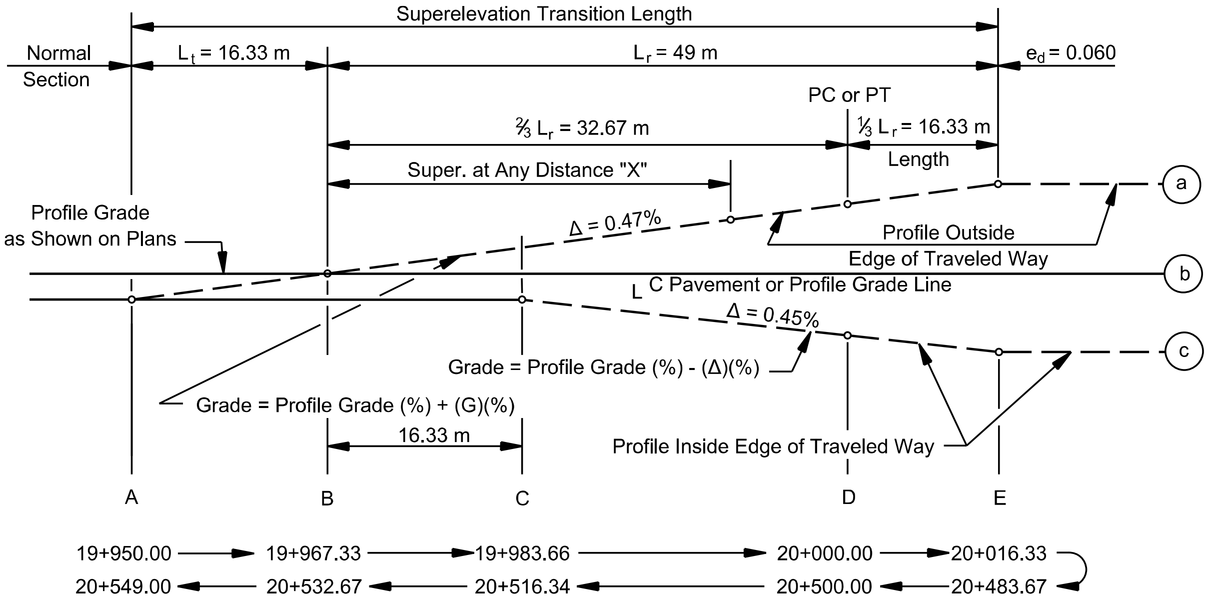


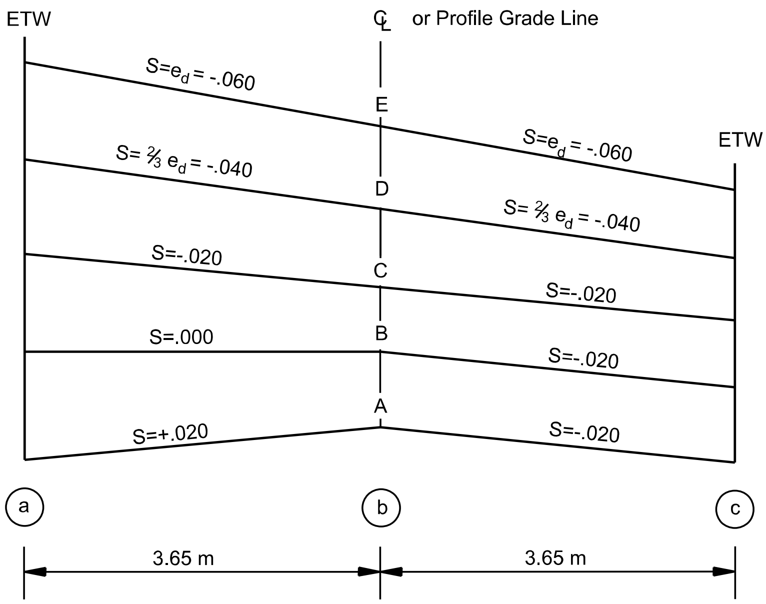
b) CASE II

* *This distance may be determined by application of G for the first curve to the increase in superelevation for the second curve ($e_{d2} – e_{d1}$).*

Note: See Section 5.3.6 for a discussion on compound curves.

Figure 5-16: Example 5.3-1





3. Determine Superelevation Runoff Length (L_r). From Table 5-9, L_r = 49 m for rotating one lane.

4. Determine Tangent Runout Length (L_t).

$$L_t = \left(\frac{e_{NC}}{e_d}\right) L_r = \left(\frac{0.020 \text{ m/m}}{0.060 \text{ m/m}}\right) 49 \text{m} = 16.33 \text{ m}$$

5. Determine Relative Gradient (Δ). The Δ values are calculated using the following equation:

$$\Delta = \frac{\text{Elev.@ X} - \text{Elev.@ Y}}{\text{Distance from Pt. X to Pt. Y}}$$

and elevation of profile gradeline = 0.0.

a. Gradient for Edge of Traveled Way at “a” from Point A to F:

$$\Delta = \frac{(0.020 \text{ m/m})(3.65 \text{m}) - (-0.06 \text{ m/m})(3.65 \text{ m})}{49 \text{ m} + 13.33 \text{ m}} = 0.0047$$

b. Gradient for Edge of Traveled Way at “c” from Point C to F:

$$\Delta = \frac{(-0.020 \text{ m/m})(3.65 \text{ m}) - (-0.060 \text{ m/m})(3.65 \text{m})}{49 \text{m} - 16.33 \text{ m}} = 0.0045$$

Example 5.3-2 (2 + 2 Dual Carriageway)

Given: Facility — 2+2 divided rural expressway
Travel lane cross slope = 2% (on tangent) = 0.02 m/m = e_{NC}
Pavement width = 7.30 m
Median width = 30 m
Design speed = 120 km/h
R = 1000 m
PC = Station 20+000
PT = Station 21+500

Problem: With the axis of rotation about the median edge, determine the applicable details for superelevation development for the horizontal curve, including:

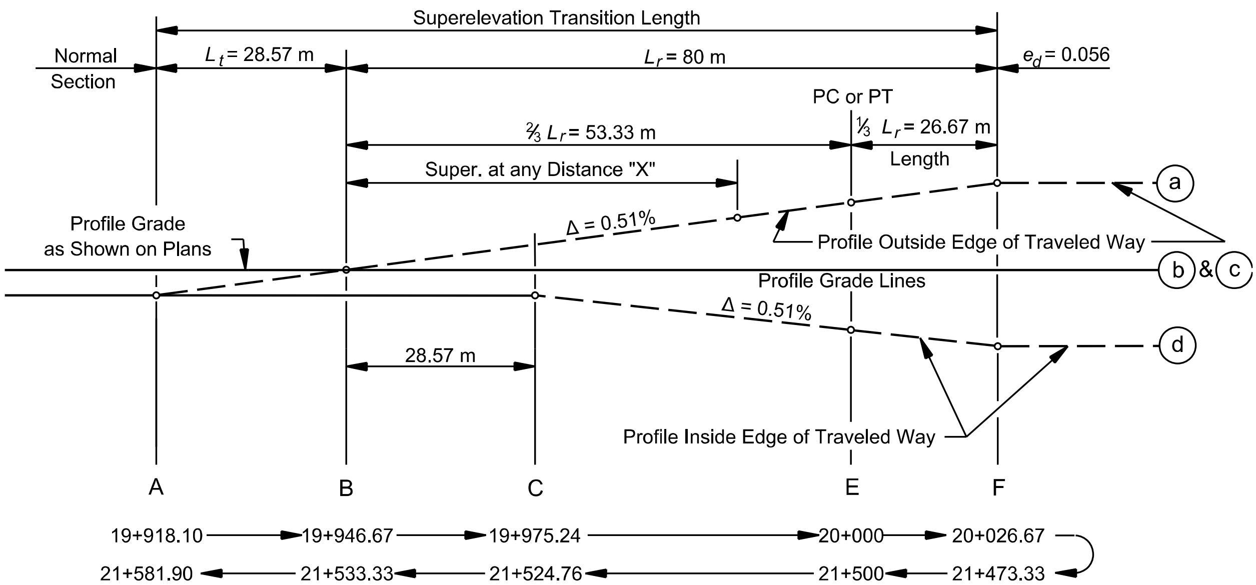
- e_{max}
- design superelevation rate, e_d
- superelevation runoff length, L_r
- tangent runout length, L_t, and
- relative gradient for the two outside edges of travelled way.

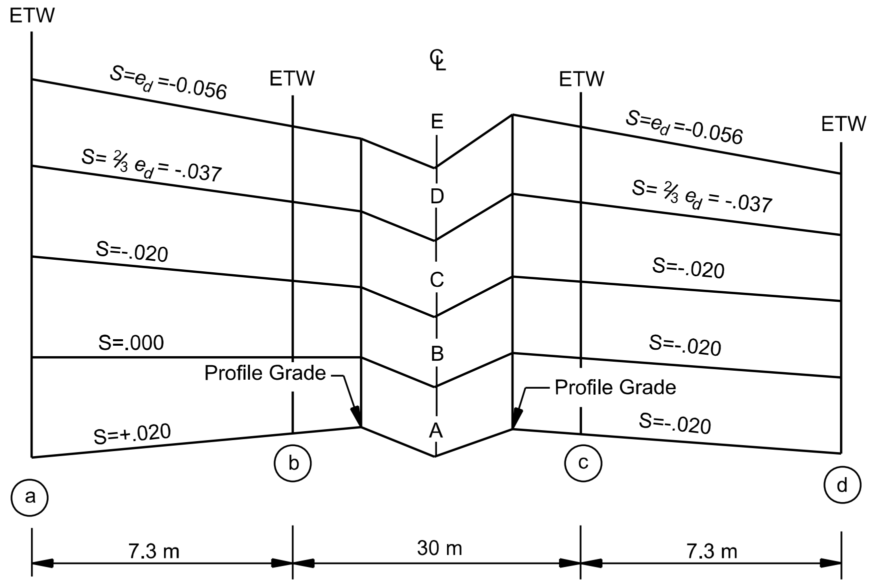
Solution: The details of the superelevated curve are determined as follows, and Figure 5-17 presents the completed example and shows all stationing.

1. Determine e_{max}. As discussed in Section 5.3.1, e_{max} = 4% to 6% for rural expressways and open-roadway conditions. For this example, use 0.06.

2. Determine Design Superelevation Rate (e_d). From Table 5-7, e_d = 0.056 m/m for radii between 980 m and 1060 m and V_d = 120 km/h.

Figure 5-17: Example 5.3-2





3. Determine Superelevation Runoff Length (L_r).

For a divided roadway with a median > 30 m, treat each direction as a separate roadway.

From Table 5-9, L_r = 80 m for rotating two lanes.

4. Determine Tangent Runout Length (L_t).

$$L_t = \left(\frac{e_{NC}}{e_d}\right) L_r = \left(\frac{0.020 \text{ m/m}}{0.056 \text{ m/m}}\right)(80 \text{ m}) = 28.57 \text{ m}$$

5. Determine Relative Gradient (Δ). The Δ values are calculated using the following equation:

$$\Delta = \frac{\text{Elev. @ X} - \text{Elev. @ Y}}{\text{Distance from Pt. X to Pt. Y}}$$

and elevation of profile gradeline = 0.0.

a. Gradient for Edge of Travelled Way at “a” from Point A to E:

$$\Delta = \frac{(0.020 \text{ m/m})(7.30 \text{ m}) - (-0.056 \text{ m/m})(7.30 \text{ m})}{80 \text{ m} + 28.57 \text{ m}} = 0.0051$$

b. Gradient for Edge of Travelled Way at “d” from Point C to E:

$$\Delta = \frac{(-0.020 \text{ m/m})(7.30 \text{ m}) - (-0.056 \text{ m/m})(7.30 \text{ m})}{80 \text{ m} - 28.57 \text{ m}} = 0.0051$$

5.4 Spiral Curves

Motor vehicles follow a transition path as they enter or leave a circular horizontal curve. The steering change and the consequent gain or loss of lateral force cannot be achieved instantly. Figure 5-18 illustrates spiral curve equations and nomenclature.

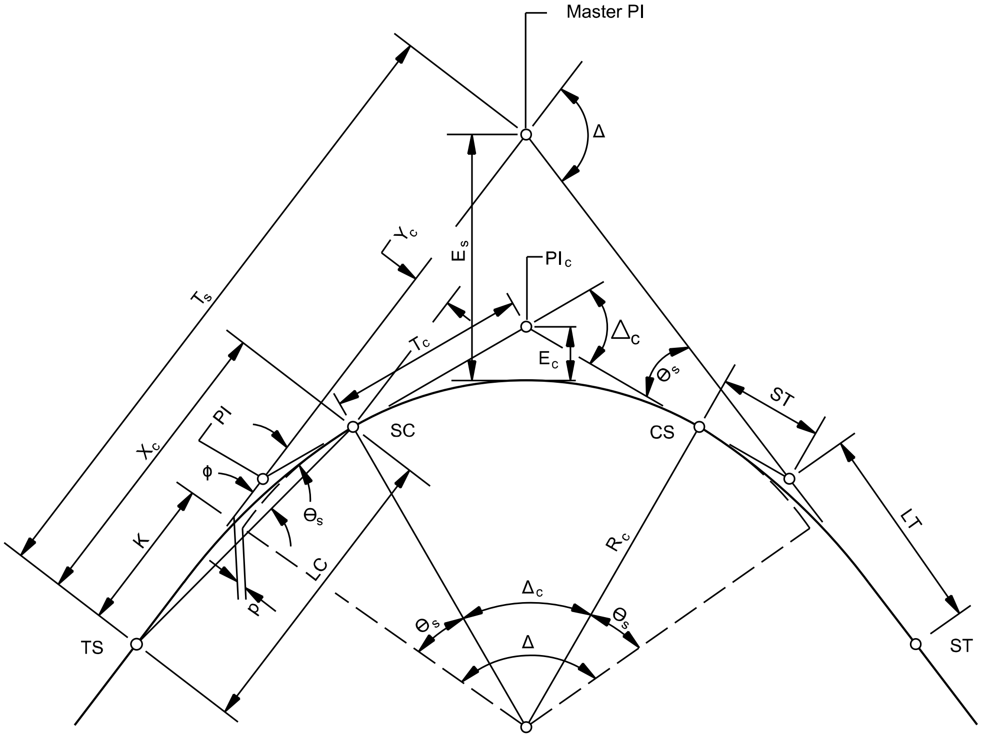
5.4.1 Application

The following lists the principal advantages of transitional spiral curves in horizontal alignment:

1. Lateral Forces. A properly designed transition curve provides a natural, easy-to-follow path for drivers such that the lateral force increases and decreases gradually as a vehicle enters and leaves a circular curve.

2. Superelevation Runoff. In spiral alignment design, superelevation runoff is affected over the whole of the transition curve. The length of the superelevation runoff should be equal to the spiral length for the tangent-to-spiral (TS) transition at the beginning and the spiral-to-curve (SC) transition at the end of the circular curve. The change in cross slope begins by removing the adverse cross slope from the lane or lanes on the outside of the curve on a length of tangent just ahead of TS (the tangent runout). Between the TS and SC, the spiral curve and the superelevation runoff are coincidental and the travelled way is rotated to reach the full superelevation at the SC. This arrangement is reversed on leaving the curve.

Figure 5-18: Spiral Curve Nomenclature



1. $\theta_s = (L_s / R_c)\ (90/\pi)$
2. $\Delta_c = \Delta - 2\theta_s$
3. $L_c = \dfrac{\Delta_c}{360}\ 2\pi R_c$
4. $T_s = (R_c + p) \tan(\Delta/2) + k$
5. $E_s = (R_c + p)\ (\sec(\Delta/2)) + p$

SPIRAL FUNCTIONS

Corrections for C in Formula $\phi = \dfrac{\theta_s}{3} - C$								
θ_s in degrees	15	20	25	30	35	40	45	50
C in minutes	0.2	0.4	0.8	1.4	2.2	3.4	4.8	6.6

6. $\phi = \dfrac{\theta}{3}$, if $\theta_s < 15°00'$ (approx. value)
7. $\varphi = \dfrac{\theta}{3} - C$, if $\theta_s \geq 15°00'$ (approx. value)
8. Deflection angle from TS or ST to any point "n" on spiral curve:

$$\phi = \frac{\theta_s}{3}\left[\frac{L'}{L_s}\right]^2$$

9. The exact values of ϕ can be determined by coordinates:

$$\tan\phi = \frac{y_c}{x_c}$$

10. $ST = \dfrac{y_c}{\sin\theta_s}$
11. $LT = x_c - \left(\dfrac{y_c}{\tan\theta_s}\right)$
12. $LC = \dfrac{x_c}{\cos\phi}$
13. $x_c = LC\cos\phi$
14. $y_c = LC\sin\phi$
15. $\theta = \dfrac{(L')^2}{{L_s}^2}\ \theta_s$

Figure 5.18: Spiral Curve Nomenclature
(*Continued*)

Master PI = Point of intersection of the main tangents.

PI_c = Point of intersection of circular curve tangents.

PI_s = Point of intersection of the main tangent and tangent of circular curve.

TS = Tangent to spiral, common point of spiral and near transition.

SC = Spiral to curve, common point of spiral and circular curve of near transition.

CS = Curve to spiral, common point of circular curve and spiral of far transition.

ST = Spiral to tangent, common point of spiral and tangent of far transition.

R_c = Radius of the circular curve.

L_s = Length of spiral.

L_c = Length of circular curve.

T_s = Tangent distance from Master PI to TS or ST, or tangent distance of completed combination of curves.

T_c = Tangent distance from SC or CS to PI_c.

E_s = External distance from Master PI to midpoint of circular curve portion

LT = Long tangent of spiral only.

ST = Short tangent of spiral only.

LC = Long chord of spiral.

p = Offset distance from the main tangent to the PC or PT of the circular curve extended.

k = Distance from TS to point on main tangent opposite the PC of the circular curve produced.

Δ = Intersection angle between main tangents of the entire curve.

Δ_c = Intersection angle between tangents at the SC and the CS or the central angle of the circular curve.

θ_s = Intersection angle between the tangent of the complete curve and the tangent at the SC, the spiral tangents intersection angle.

ϕ = Deflection angle from main tangent at TS to SC along the line of the long chord.

$x_c y_c$ = Coordinates of SC from the TS.

L' = Length of spiral arc from the TS to any point on the spiral.

θ = The central angle of spiral arc L'. θ equals θ_s when L' equals L_s.

Source: (3)

3. Travelled Way Widening. A spiral transition curve also facilitates the transition in width where the travelled way is widened on a circular curve. Use of spiral transitions provides flexibility in accomplishing the widening of sharp curves.

4. Aesthetics. The appearance of the roadway or street is enhanced by spiral transition curves. The use of spiral transitions avoids noticeable breaks in the alignment as perceived by drivers at the beginning and end of circular curves.

Table 5-12 presents guidelines on the maximum radii for where spiral curves may be considered. The operational advantages of spiral transition curves with radii greater than those shown in Table 5-12 are negligible.

Table 5-12: Guidelines for Spiral Curves

Design Speed (km/h)	Maximum Radius (m)
80	379
90	480
100	592
110	716
120	852
130	1000
140	1160

Notes:
1. *Spiral curves are typically only used on freeways, expressways, and rural arterials.*
2. *Do not use spiral curves on bridges.*
3. *The benefits of spiral curve transitions are generally negligible for larger radii.*
4. *Maximum radius for use of a spiral is based on a minimum lateral acceleration rate of 1.3 m/s^2.*

Source: (1)

5.4.2 Length of Spiral

Generally, the Euler spiral, which is also known as the clothoid, is used in the design of roadway spiral transition curves. The radius varies from infinity at the tangent end of the spiral to the radius of the circular arc at the end that adjoins that circular arc. By definition, the radius of curvature at any point on an Euler spiral varies inversely with the distance measured along the spiral.

5.4.2.1 Minimum Length of Spiral

A minimum length of spiral is generally based on consideration of driver comfort and shifts in the lateral position of vehicles. Criteria based on driver comfort are intended to provide a spiral length that allows for a comfortable increase in lateral acceleration as a vehicle enters a curve. The criteria based on lateral shift are intended to ensure that a spiral curve is sufficiently long to provide a shift in a vehicle's lateral position within its lane that is consistent with that produced by the vehicle's natural spiral path. Use these two criteria together to determine the minimum length of spiral. The minimum length of spiral should be the larger result of either Equation 5.6 or Equation 5.7.

$L_{s,\,min}$ should be the larger of:

$$L_{s,min} = 0.0214 \frac{V^3}{RC}$$

Equation 5.6 Minimum Spiral Length

$$L_{s,min} = \sqrt{24(p_{min})R}$$

Equation 5.7: Minimum Spiral Length

where: $L_{s,min}$ = minimum length of spiral, m
p_{min} = minimum lateral offset between tangent and circular curve (0.20 m)
R = radius of circular curve, m
V = design speed, km/h
C = maximum rate of change in lateral acceleration (1.2 m/s^3)

Use a value of 0.20 m for p_{min}. This value is consistent with the lateral shift that occurs as a result of the natural steering behaviour of most drivers. The minimum value for C is 1.2 m/s^3. The use of lower values will yield longer, more comfortable spiral curve lengths; however, these lengths would not represent the minimum length consistent with driver comfort.

5.4.2.2 Maximum Length of Spiral

Spirals should not be so long (relative to the length of the circular curve) that drivers are misled about the sharpness of the approaching curve. A conservative maximum length of spiral that should minimise the likelihood of such concerns can be computed as:

$$L_{s,max} = \sqrt{24\left(p_{max}\right)R}$$

Equation 5.8: Maximum Length of Spiral

where: $L_{s,max}$ = maximum length of spiral, m
p_{max} = maximum lateral offset between tangent and circular curve (1.0 m)
R = radius of circular curve, m

Use a value of 1.0 m for p_{max}. This value is consistent with the maximum lateral shift that occurs as a result of the natural steering behaviour of most drivers. It also provides a reasonable balance between spiral length and curve radius.

5.4.2.3 Desirable Length of Spiral

Desirable lengths of spiral transition curves are shown in Table 5.13.These lengths correspond to 2.0 s of travel time at the design speed of the roadway. This travel time has been found to be representative of the natural spiral path for most drivers. Theoretical considerations suggest that significant deviations from these lengths tend to increase the shifts in the lateral position of vehicles within a lane that may precipitate encroachment on an adjacent lane or shoulder.

If the desirable spiral curve length shown in Table 5.13 is less than the minimum spiral curve length determined from Equation 5.6 and Equation 5.7, the minimum spiral curve length should be used in design

Table 5-13: Desirable Length of Spiral Curve Transition

Design Speed (km/h)	Spiral Length (m)
20	11
30	17
40	22
50	28
60	33
70	39
80	44
90	50
100	56
110	61
120	67
130	72
140*	78

**Estimated by DMT.*

Source: (1)

5.4.3 Length of Superelevation Runoff

In transition design with a spiral curve, the superelevation runoff should be accomplished over the length of spiral. The length of runoff is applicable to all superelevated curves, and that this value is used for minimum lengths of spiral. In this manner, the length of spiral should be set equal to the length of runoff. The change in cross slope begins by introducing a tangent runout section just in advance of the spiral curve. Full attainment of superelevation is then accomplished over the length of the spiral. In this design, the whole of the circular curve has full superelevation.

5.4.4 Limiting Superelevation Rates

One consequence of equating runoff length to spiral length is that the resulting relative gradient of the pavement edge may exceed the values listed in Table 5-10. However, small increases in gradient have not been found to have an adverse effect on comfort or appearance.

5.4.5 Length of Tangent Runout

The tangent runout length for a spiral curve transition design is based on the same approach used for the tangent-to-curve transition design. Specifically, a smooth edge of pavement profile is desired so that a common edge slope gradient is maintained throughout the superelevation runout and runoff sections. Based on this rationale, Equation 5.14 can be used to compute the tangent runout length:

$$L_t = \frac{e_{NC}}{e_d} L_s$$

Equation 5.9: Length of Tangent Runout

where: L_t = length of tangent runout, m
L_S = length of spiral, m
e_d = design superelevation rate, percent

e_{NC} = normal cross slope rate, percent

The tangent runout lengths obtained from Equation 5.9 tend to be longer than desirable for combinations of low superelevation rate and high design speeds. Long runout lengths may present safety problems when there is insufficient profile grade to provide adequate pavement surface drainage. Many problems can be avoided when the minimum profile grade criteria described in Chapter 3 “Cross Section Elements” are applied to the spiral curve transition.

5.5 Horizontal Sight Offset

5.5.1 Overview

Sight obstructions on the inside of a horizontal curve are obstacles of considerable length that interfere with the line of sight on a continuous basis. These include roadside barriers, walls, cut slopes, and buildings. In general, point obstacles (e.g. traffic signs, utility poles) are not considered sight obstructions on the inside of horizontal curves. The designer must examine each curve individually to determine whether it is necessary to remove an obstruction or adjust the horizontal alignment to obtain the required sight distance.

5.5.2 Equations

Where the length of curve (L) is greater than the sight distance (S) used for design (i.e. stopping sight distance, decision sight distance, passing sight distance), the needed clearance on the inside of the horizontal curve is calculated using the following equation:

$$\mathbf{HSO} = \mathbf{R}\left(\mathbf{1} - \mathbf{cos}\left[\frac{\mathbf{28.65\ S}}{\mathbf{R}}\right]\right)$$

Equation 5.10: Horizontal Sight Offset (L > S)

where: HSO = Middle ordinate or horizontal sightline offset from the centre of the inside travel lane to the obstruction, m

R = Radius of curve, m

S = Sight distance, m

Example 5.5-1 (Figure 5-19) illustrates the use of Equation 5.10. Example 5.5-1 (Figure 5-19) also illustrates how to determine horizontal clearances for the entering/exiting portions of a curve.

Where the length of curve is less than the sight distance used in design, the HSO value from Equation 5.10 will never be reached. As an approximation, the horizontal clearance for these curves should be determined as follows:

Step 1: For the given R and S, calculate HSO assuming L > S.

Step 2: The maximum HSO′ value will be needed at a point of L/2 beyond the PC. HSO′ is calculated using Equation 5.11. Example 5.5-2 (Figure 5-20) illustrates the use of Equation 5.11.

$$\frac{HS\,O'}{HSO} = \frac{1.2L}{S}$$

$$HS\,O' = \frac{1.2(L)(HSO\quad)}{S}$$

Equation 5.11: Horizontal Sight Distance (L < S)

At a minimum, the designer needs ensure stopping sight distance (SSD) is available throughout the horizontal curve. In some situations, the designer may also want to check the middle ordinate for passing sight distances or decision sight distances.

Figure 5-21 provides the horizontal clearance criteria (i.e. middle ordinate horizontal sightline offset) for various combinations of SSD (see Chapter 4 “Sight Distance”) and curve radii for passenger cars on level grades. Where the grade is greater than 3%, the road designer should also check the middle ordinate using the SSD values adjusted for grades.

<u>Example 5.5-1</u>

Given: Design Speed = 90 km/h
R = 300 m
Level Grade

Problem: Determine the horizontal clearance requirements for a horizontal curve on a two-lane highway using stopping sight distance (SSD).

Solution: Table 4-1 yields a SSD = 160 m. Using Equation 5.10 for horizontal clearance L > S):

$$HSO = 300\left(1 - \cos\left[\frac{(28.65)(160)}{300}\right]\right) = 11\text{ m}$$

Figure 5-19 also illustrates the horizontal clearance requirements for the entering and exiting portion of the horizontal curve.

Figure 5-19: Sight Clearance Requirements for Horizontal Curves (L > S)

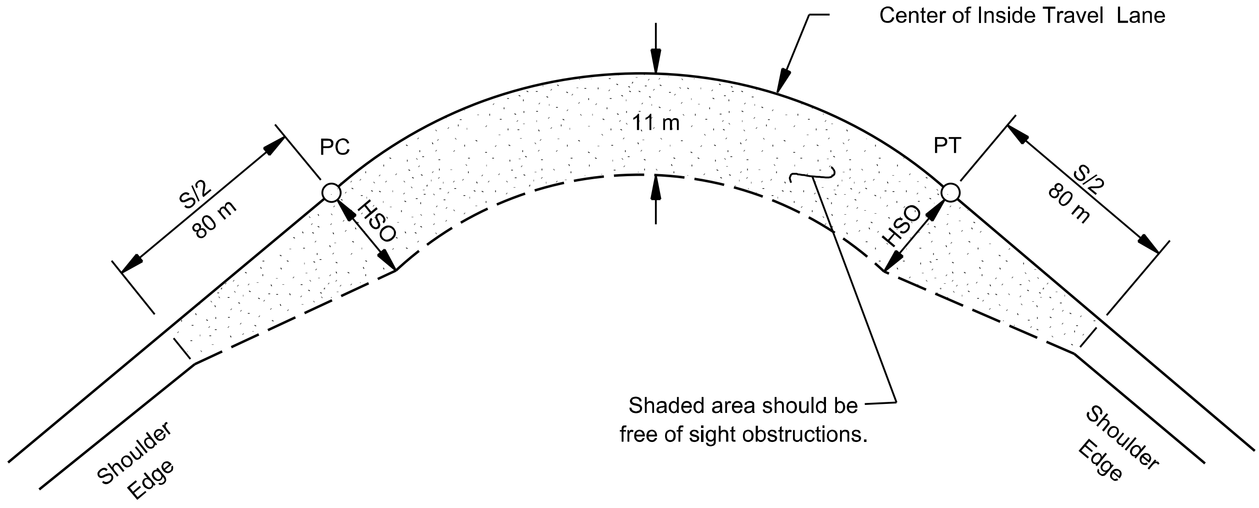


Figure 5-20: Sight Clearance Requirements for Horizontal Curves (S > L)

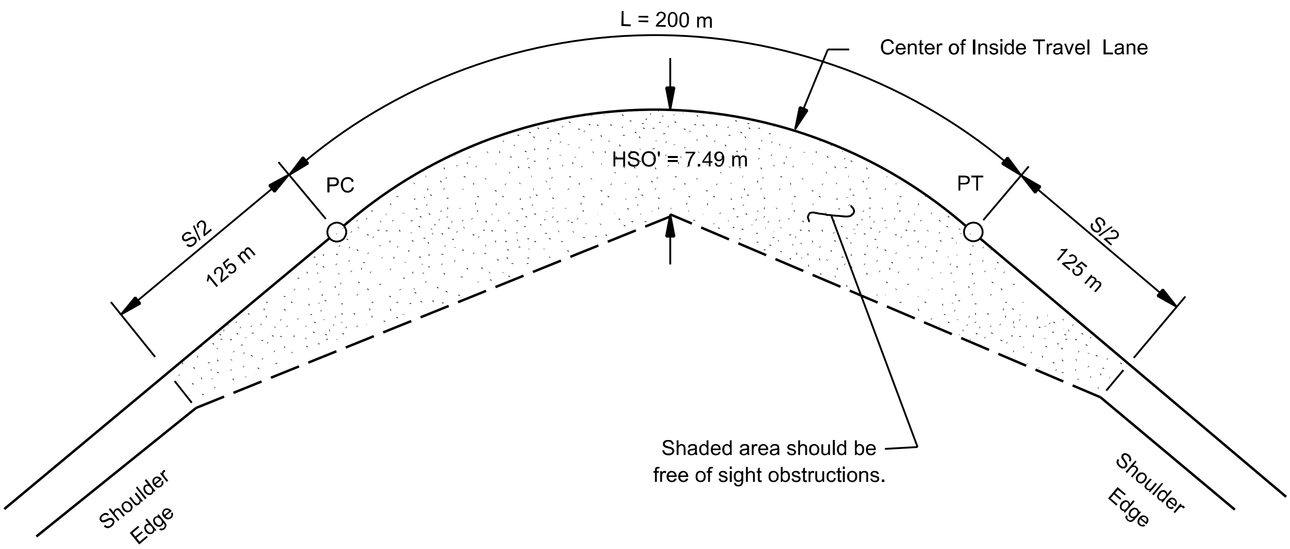
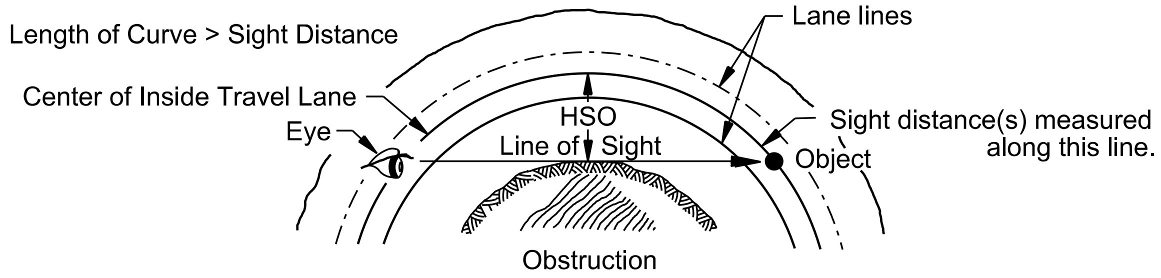
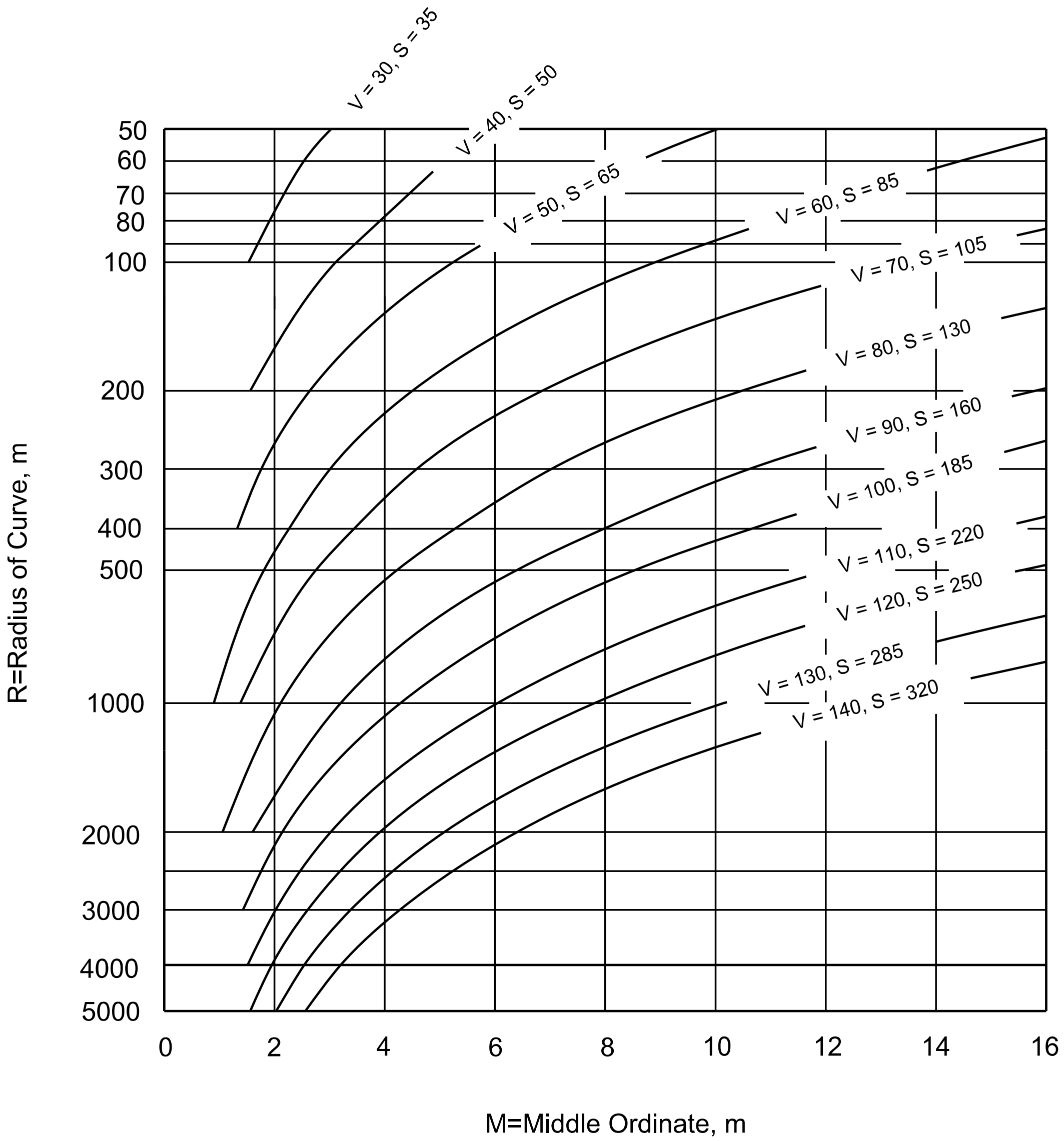


Figure 5-21: Sight Distance at Horizontal Curves (SSD)





V=Design Speed (km/h)

Example 5.5-2

Given: Design speed = 120 km/h
R = 1000 m
L = 200 m
See Figure 5-20.

Problem: Determine the horizontal sightline offset requirements for the horizontal curve on a two-lane roadway assuming passenger car SSD.

Solution: Table 4-1 yields a SSD = 250 m. Therefore, L < S (200 m < 250 m), and the horizontal clearance is calculated from Equations 5.10 and 5.11 as follows:

$$\text{HSO (L} > \text{S)} = 1000\left[1 - \cos\frac{(28.65)(250)}{1000}\right] = 7.80\text{ m}$$

$$\text{HSO}'\ (\text{L} < \text{S}) = \frac{1.2(200)(7.80)}{250}$$

$$\text{HSO}' = 7.49\text{ m}$$

Therefore, provide a minimum clearance of 7.49 m at a distance of L/2 = 100 m beyond the PC. The obstruction-free triangle around the horizontal curve would be defined by HSO′ (7.49 m) at L/2 and by points at the shoulder edge at S/2 = 125 m before the PC and beyond the PT.

5.5.3 Eye and Object Heights

For application, the height of eye is 1.08 m and the height of object is 0.6 m. Both the eye and object are assumed to be in the centre of the inside travel lane. The line-of-sight intercept with the obstruction is at the midpoint of the sightline and 0.84 m above the centre of the inside lane.

5.6 Curve Widening

5.6.1 General Theory

Travelled way on horizontal curves may be widened to create operating conditions on curves that are comparable to those on tangents. On modern roadways and streets with 3.65 m travel lanes and high-type alignment, the need for widening generally is not required in spite of high speeds, but for some conditions of speed, curvature, and width it may be appropriate to widen the travelled way.

The amount of widening of the travelled way on a horizontal curve is the difference between the width needed on the curve and the width used on a tangent.

$$\boxed{w = W_c - W_n}$$

Equation 5.12: Curve Widening

where: w = widening of travelled way on curve, m
W_c = width of travelled way on curve, m (Equation 5.13)
W_n = width of travelled way on tangent, m

The travelled way width needed on a curve (W_c) has several components related to operation on curves, including: the track width of each vehicle meeting or passing (U) Equation 5.14, the lateral clearance for each vehicle (C), width of front overhang of the vehicle occupying the inner lane or lanes (F_A) Equation 5.15, and a width allowance for the difficulty of driving on curves (Z) Equation 5.16. These components are illustrated in Figure 5-22.

$$W_c = N(U + C) + (N - 1)F_A + Z$$

Equation 5.13

$$U = u + R - \sqrt{R^2 - \sum L_i^2}$$

Equation 5.14

$$F_A = \sqrt{R^2 + A(2L + A)} - R$$

Equation 5.15

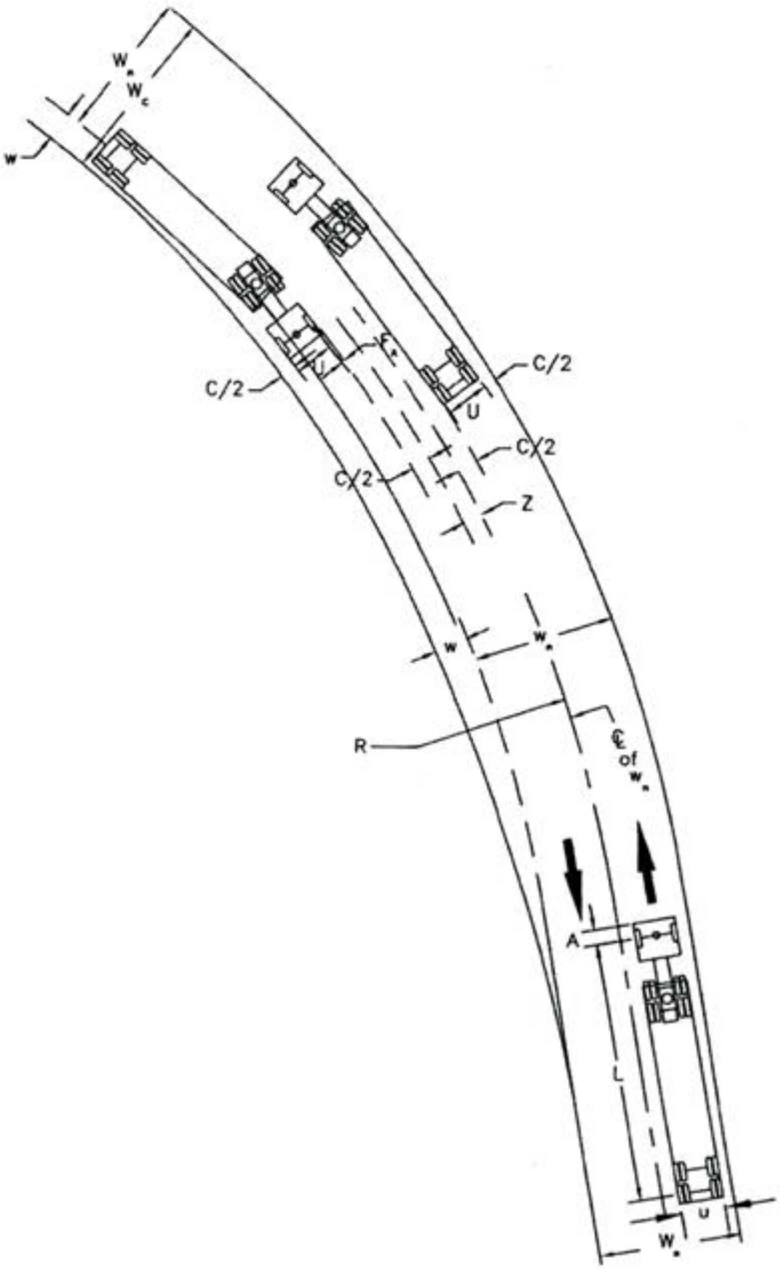
$$Z = 0.1\left(V / \sqrt{R}\right)$$

Equation 5.16

where:

U = track width of design vehicle (out-to-out tires) on curves, m
μ = track width on tangent (out-to-out tires), m
R = radius of curve or turn, m
L_i = wheelbase of design vehicle between consecutive axles (or sets of tandem axles) and articulation points, m
F_A = width of front overhang of inner-lane vehicle, m
A = from overhang of inner lane vehicle, m
L = wheelbase of single unit or tractor, m
Z = extra width allowance, m
V = design speed of the roadway, km/h
W_c = width of travelled way on curve, m
N = number of lanes
C = lateral clearance, m

Figure 5-22: Widening Components on Open Roadway Curves



Source: (1)

The lateral clearance, C, provides clearance between the edge of the travelled way and nearest wheel path and for clearances between vehicles passing or meeting. For design, assume C = 0.9 m.

5.6.2 Design Values

Table 5-14 provides “w” values based on the WB-20 design vehicle. The WB-20 design vehicle will generally be the largest vehicle allowed on the roadway; consequently, will yield the greatest travelled way width required. The values in Table 5-14 are applicable to two-lane roadways, as well as, dual carriageways.

Widening is costly and very little is actually gained from a small amount of widening. Do not consider widening if the widening is less than 0.6 m. Values less than 0.6 m in Table 5-14 only apply for multilane roadways (i.e. for two-lane roads the values above the heavy line should be disregarded).

Table 5-14 applies to open-roadway curves. For turning roadways and other curves at intersections, the criteria for the roadway design widths are different. Chapter 10 “Intersections” presents the design criteria for these conditions.

5.6.3 Application

Widening should transition gradually on the approaches to the curve to provide a reasonably smooth alignment of the edge of the travelled way and to fit the paths of vehicles entering or leaving the curve. The principal points of concern in the design of curve widening, which apply to both ends of roadway curves, are presented below:

1. Location of Widening. On simple curves, only apply the widening on the inside edge of the travelled way. On curves designed with spirals, widening may be applied on the inside edge or divided equally on either side of the centreline. In the latter method, extension of the outer-edge tangent avoids a slight reverse curve on the outer edge. In either case, the final marked centreline, and preferably any central longitudinal joint, should be placed midway between the edges of the widened travelled way.

2. Transition. Transition the curve widening gradually over a length sufficient to make the whole travelled way fully usable. Although a long transition is desirable for traffic operation, it may result in narrow pavement slivers that are difficult and expensive to construct. Preferably, widening should transition over the superelevation runoff length, but shorter lengths are sometimes used. Changes in width normally should be effected over a distance of 30 m to 60 m.

3. Aesthetics. From the standpoints of usefulness and appearance, the edge of the travelled way through the widening transition should be a smooth curve. Do not use a tangent transition edge. The transition ends should avoid an angular break at the pavement edge.

4. Distribution. On roadway alignment without spirals, smooth and fitting alignment results from attaining widening with one-half to two-thirds of the transition length along the tangent and the balance along the curve. This is consistent with a common method for attaining superelevation. The inside edge of the travelled way may be designed as a modified spiral, with control points determined by either the width/length ratio of a triangular wedge, by calculated values based on a parabolic or cubic curve, or by a larger radius (compound) curve. Otherwise, it may be aligned by eye in the field. On roadway alignment with spiral curves, the increase in width is usually distributed along the length of the spiral.

Table 5-14: Calculated and Design Values for Travelled Way Widening on Open Roadway Curves (Two-Lane Roadways, One-Way or Two-Way)

Radius of Curve (m)	Roadway Width = 7.3 m						Roadway Width = 6.6 m						Roadway Width = 6.0 m					
	Design Speed (km/h)						Design Speed (km/h)						Design Speed (km/h)					
	50	60	70	80	90	100	50	60	70	80	90	100	50	60	70	80	90	100
3000	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.3	0.3	0.3	0.3	0.3	0.5	0.6	0.6	0.6	0.6	0.6
2500	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3	0.3	0.3	0.3	0.4	0.6	0.6	0.6	0.6	0.6	0.7
2000	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3	0.3	0.4	0.4	0.4	0.6	0.6	0.6	0.7	0.7	0.7
1500	0.0	0.0	0.0	0.0	0.0	0.1	0.3	0.4	0.4	0.4	0.4	0.5	0.6	0.7	0.7	0.7	0.7	0.8
1000	0.0	0.1	0.1	0.1	0.2	0.2	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9
900	0.1	0.1	0.1	0.2	0.2	0.2	0.5	0.5	0.5	0.6	0.6	0.6	0.8	0.8	0.8	0.9	0.9	0.9
800	0.1	0.1	0.2	0.2	0.2	0.3	0.5	0.5	0.6	0.6	0.6	0.7	0.8	0.8	0.9	0.9	0.9	1.0
700	0.3	0.3	0.3	0.4	0.4	0.4	0.7	0.7	0.7	0.8	0.8	0.8	1.0	1.0	1.0	1.1	1.1	1.1
600	0.3	0.4	0.4	0.4	0.5	0.5	0.7	0.8	0.8	0.9	0.9	0.9	1.0	1.1	1.1	1.1	1.2	1.2
500	0.4	0.4	0.5	0.5	0.6	0.6	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.3	1.3
400	0.5	0.6	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.5
300	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.4	1.4	1.4	1.5	1.5	1.6	1.7	1.7
250	1.0	1.1	1.1	1.2	1.2		1.3	1.4	1.4	1.5	1.5		1.6	1.7	1.7	1.8	1.8	
200	1.2	1.3	1.4	1.4			1.6	1.7	1.8	1.8			1.9	2.0	2.1	2.1		
150	1.7	1.8	1.9	2.0			2.0	2.1	2.2	2.3			2.3	2.4	2.5	2.6		
140	1.8	1.9					2.2	2.3					2.5	2.6				
130	2.0	2.					2.4	2.4					2.7	2.7				
120	2.1	2.2					2.5	2.6					2.8	2.9				
110	2.4	2.5					2.7	2.8					3.0	3.1				
100	2.6	2.7					3.0	3.1					3.3	3.4				
90	2.8						3.2						3.5					
80	3.2						3.6						3.9					
70	3.7						4.0						4.3					

Notes:

1. *Values shown are for a WB-20 design vehicle and represent widening in metres.*
2. *Values less than 0.6 m may be disregarded.*
3. *For 1+2 roadways, multiply above values by 1.5.*
4. *For 2+2 roadways, multiply above values by 2.0.*

5.7 General Controls for Horizontal Alignment

The design of horizontal alignment involves, to a large extent, complying with specific limiting criteria. These include minimum radius, superelevation rates, and sight distance around curves. In addition, the designer should adhere to certain design principles and controls that will determine the overall safety of the facility and will enhance the aesthetic appearance of the roadway. These design principles include:

1. Consistency. The alignment should be consistent. Sharp curves at the ends of long tangents and sudden changes from gentle to sharply curving alignment should be avoided; see Figure 5-23(a).

2. Directional. The alignment should be as directional as possible consistent with physical and economic constraints. On divided roadways, a flowing line that conforms generally to the natural contours is preferable to one with long tangents that slash through the terrain. Directional alignment will be achieved by using the smallest practical central angles.

3. Use of Minimum Radii. Avoid the use of minimum radii, if practical, especially in level terrain; see Figure 5-23(a).

4. High Fills. Avoid sharp curves on long, high fills. Under these conditions, it is difficult for drivers to perceive the extent of horizontal curvature.

5. Compound Curves. Do not use compound curves on the roadway mainline.

6. Alignment Reversals. Avoid abrupt reversals in alignment (reverse curves). Desirably, provide a sufficient tangent distance between the curves to ensure proper superelevation transitions for both curves and to allow time for the motorist to perceive the next decision point. Typically, 2 seconds of travel time; see Figure 5-23(b).

7. Broken-Back Curvature. Avoid where possible. This arrangement is not aesthetically pleasing, violates driver expectancy, and creates undesirable superelevation development requirements; see Figure 5-23(c).

8. Coordination with Natural/Man-Made Features. The horizontal alignment should be properly coordinated with the natural topography, available right-of-way, utilities, roadside development, and natural/man-made drainage patterns.

9. Environmental Impacts. Horizontal alignment should be properly coordinated with environmental impacts.

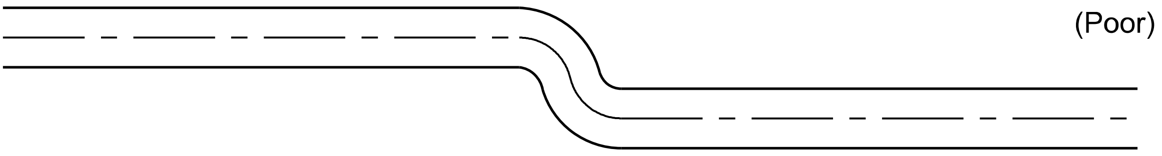
10. Intersections. Horizontal alignment through intersections may present special problems (e.g., intersection sight distance, superelevation development crossover crowns). See Chapter 10 “Intersections” for the design of intersections.

11. Coordination with Vertical Alignment. Chapter 6 “Vertical Alignment” discusses general design principles for the coordination between horizontal and vertical alignment.

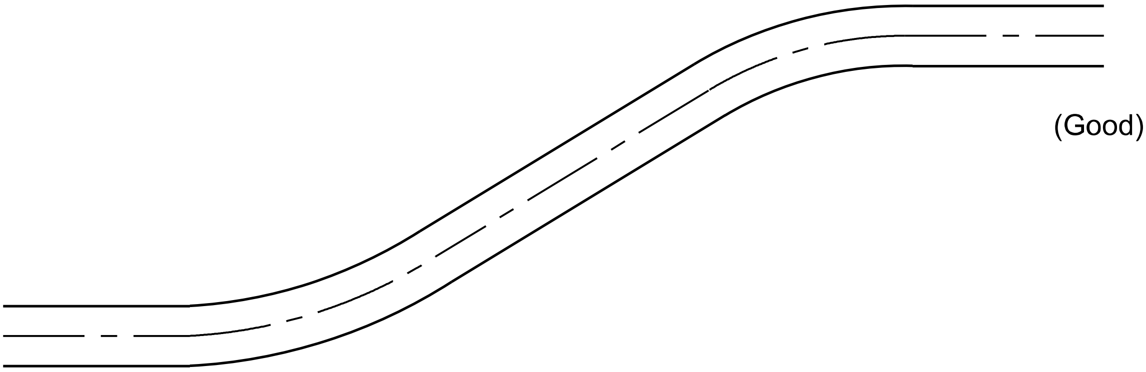
12. Bridges. Horizontal alignment must be coordinated with the location of bridges. The need for curvature and superelevation development should be evaluated for each bridge location.

Chapter 6 “Vertical Alignment” provides illustrations of good and poor coordination horizontal and vertical alignments.

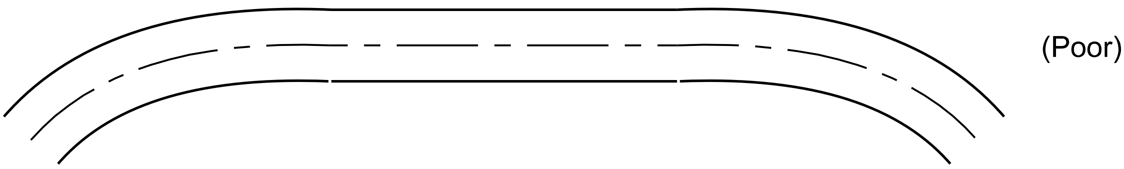
Figure 5-23: Horizontal Curve Coordination



a) Curve with Short Radii



(b) Alignment Reversal



(c) Broken Back Curve

6 VERTICAL ALIGNMENT

6.1 Overview

This chapter provides guidance on vertical alignment elements that include maximum and minimum grades, critical length of grade, truck-climbing lanes, vertical curvature, vertical clearances, aesthetics, profile gradelines, and emergency escape ramps.

6.1.1 Vertical Alignment Position with Respect to Cross Section

The grade line (defined by the master string) should generally coincide with the axis of rotation for superelevation and should relate to the cross section as follows:

Undivided Roadways - The grade line should coincide with the roadway centreline.

Primary Roadway Connections and Ramps - The grade line may be positioned at either edge of the travelled way, or centreline if multi-lane.

Divided Roadways - The grade line may be positioned at either the median centreline or at the ultimate median edge of travelled way. The former case is appropriate for paved medians 9m wide or less. The latter case is appropriate when:

1. The median edges of travelled way of the two roadways are at equal elevation.
2. The roadways are at different elevations.
3. The median width is non-uniform.

6.2 Grades and Terrain

6.2.1 Terrain/Topography

The topography of the land traversed has an influence on the alignment of roads and streets. Topography affects horizontal alignment, but has an even more pronounced effect on vertical alignment. To characterise variations in topography, topography is generally separate into three classifications according to terrain:

1. Level. In level terrain, sight distances, as governed by both horizontal and vertical restrictions, are generally long or can be made so without construction difficulty or major expense.

2. Rolling. In rolling terrain, natural slopes consistently rise above and fall below the road or street grade and occasional steep slopes offer some restrictions to normal horizontal and vertical roadway alignment.

3. Mountainous. In mountainous terrain, longitudinal and transverse changes in the elevation of the ground with respect to the road or street are abrupt, and benching and side hill excavation are frequently needed to obtain acceptable horizontal and vertical alignment.

In mountainous areas, the presence of significant lengths of grade, coupled with the potential difficulty of accommodating high design speeds within the topographical constrains, often necessitates the selection of lower design speeds. Although the terrain in the Emirati of Abu Dhabi is relatively level, there are some hilly places where the design speed may need to be reduced below the minimum values.

Terrain classifications pertain to the general character of a specific route corridor. Routes in valleys, passes, or mountainous areas that have all the characteristics of roads or streets traversing level or rolling terrain should be classified as level or rolling.

6.2.2 Maximum Grades

Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads,” and Chapter 9 “Urban Streets” present criteria for maximum grades based on functional classification, urban/rural location, type of terrain, and design speed. Wherever practical, use grades flatter than the maximum. Only use the maximum grades where it is absolutely necessary.

6.2.3 Minimum Grades

The following provides criteria for minimum grades:

1. Uncurbed Roadways. It is acceptable to provide a minimum longitudinal gradient of approximately 0.0% without affecting the requirements in Clause 6.4.1.5, but only if the roadway will not be curbed in the future. Ensure that the pavement has adequate cross slopes and that the flow lines of the outside ditches have adequate drainage.

2. Curbed Streets. Curbed Streets: The median edge or centerline profile on roadways and streets with curb and gutter desirably should have a longitudinal gradient of at least 0.2% for curbed local roads and collectors, 0.3% for curbed arterials, freeways and expressways.

6.2.4 Trucks

Steep grades affect truck speeds and overall capacity. Compared to passenger cars, trucks typically accelerate more slowly from a stop condition and experience greater difficulty in maintaining desirable speeds on long, steep upgrades. Speed differential on roadways with steep grades can contribute to safety and operational problems. Trucks lose speed on steep, ascending grades and may be unable to reach full roadway speed until they have passed the crest of the steep grade. Truck drivers may also choose to descend grades at lower speeds to maintain better control of their vehicles. Vehicles behind them are slowed, lowering operations, which may contribute to rear-end crashes and, in some cases, risky passing manoeuvres.

For these reasons, it is desirable to provide the flattest grades practicable. Grades above 2% may affect truck traffic depending on the length of grade. The designer should consider the following grade guidelines when designing vertical grades for trucks:

- Maximum grade for level terrain: 2% – 3%
- Maximum grade for rolling terrain: 4%

These modest grades can still have an impact on truck operating speed. Attempts should be made to minimise the impact of grade and length of grade on truck operating speed.

6.2.5 Critical Length of Grade

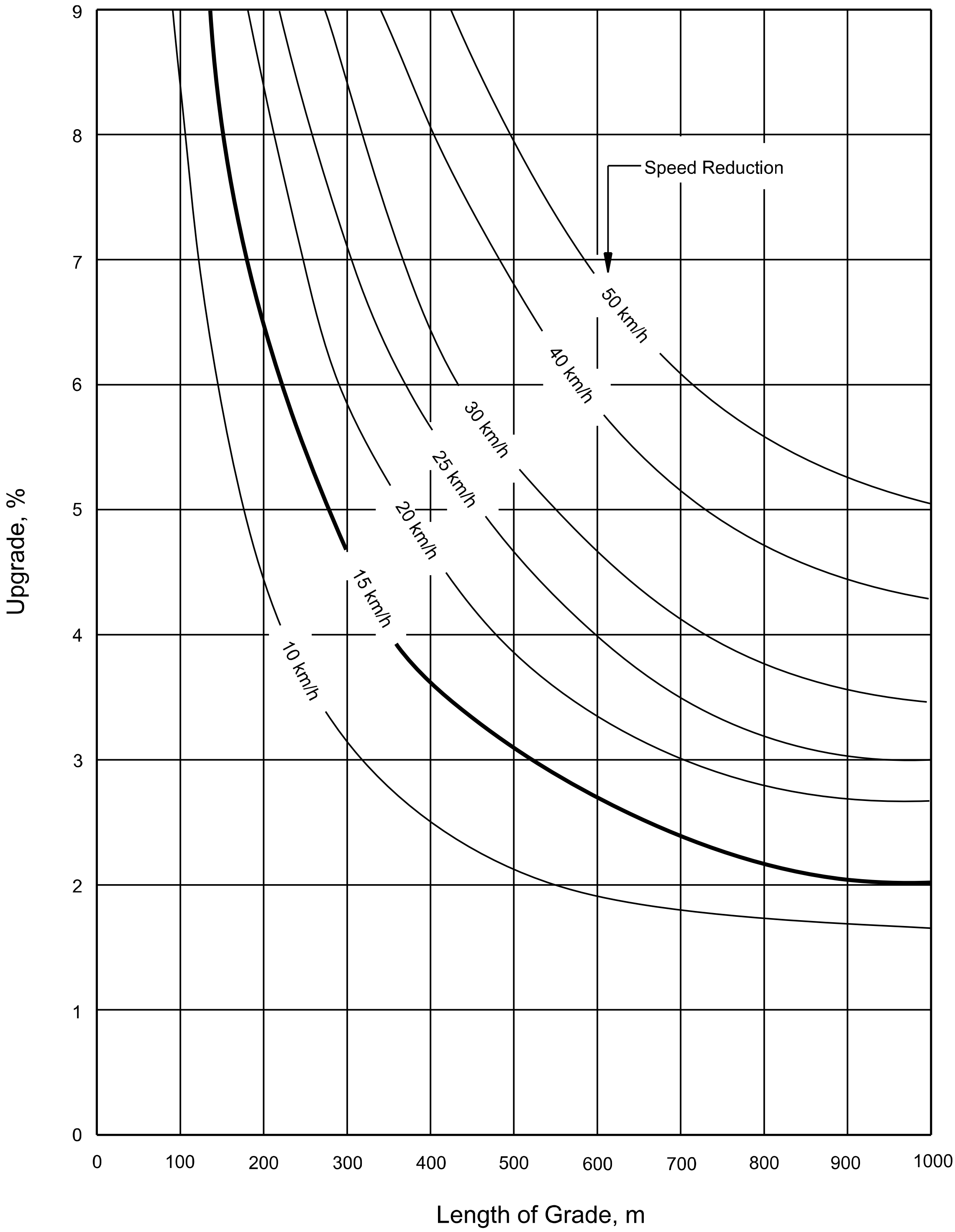
The critical length of grade is the maximum length of a specific upgrade on which a truck can operate without an unreasonable reduction in speed. The roadway gradient, in combination with the length of the grade, will determine the truck speed reduction on upgrades.

The following will apply to the critical length of grade:

1. Design Vehicle. Figure 6-1 presents the critical length of grade for a 120 kg/kW truck. This vehicle is representative of size and type of a heavy vehicle normally used in Abu Dhabi.

2. Criteria. Figure 6-1 provides the critical lengths of grade for a given percent grade and acceptable truck speed reduction. Although this figure is based on an initial truck speed of 110 km/h, it applies to all design or posted speeds. For design purposes, use the 15 km/h speed reduction curve in Figure 6-1 to determine if the critical length of grade is exceeded.

Figure 6-1: Critical Length of Grade (Trucks)



Notes:

1. *Typically, the 15 km/h curve will be used.*
2. *See Section 6.2.5 on how to use this figure.*
3. *This figure is based on a 120 kg/kW truck with initial speed of 110 km/h. However, it may be used for any design or posted speed.*

Source: (1)

3. Momentum Grades. Where an upgrade is preceded by a downgrade, trucks will often increase their speed to ascend the upgrade. A speed increase of 10 km/h on moderate downgrades (3% to 5%) and 15 km/h on steeper downgrades (6% to 8%) of sufficient length are reasonable adjustments to the initial speed. This assumption allows the use of a higher speed reduction curve in Figure 6-1. However, the designer should also consider that these speed increases may not always be attainable. If traffic volumes are sufficiently high, a truck may be behind another vehicle when descending the momentum grade thereby restricting the increase in speed. Therefore, only consider these increases in speed if the roadway has a Level of Service equal to C or better.

4. Measurement. Vertical curves are part of the length of grade. Figure 6-2 illustrates how to measure the length of grade to determine the critical length of grade.

5. Application. If the critical length of grade is exceeded, flatten the grade, if practical, or evaluate the need for a truck-climbing lane. Typically, only two-lane roadways have operational problems that require truck-climbing lanes.

6. Roadway Types. The critical-length-of-grade criterion applies equally to two-lane (single carriageway) or multilane roadways (dual carriageway), and applies equally to urban and rural facilities.

Examples 6.2-1 and 6.2-2 illustrate the use of Figure 6-1 to determine the critical length of grade. Example 6.2-3 illustrates how to use both Figure 6-1 and Figure 6-2. In the examples, the use of subscripts 1, 2, etc., indicate the successive gradients and lengths of grade on the roadway segment.

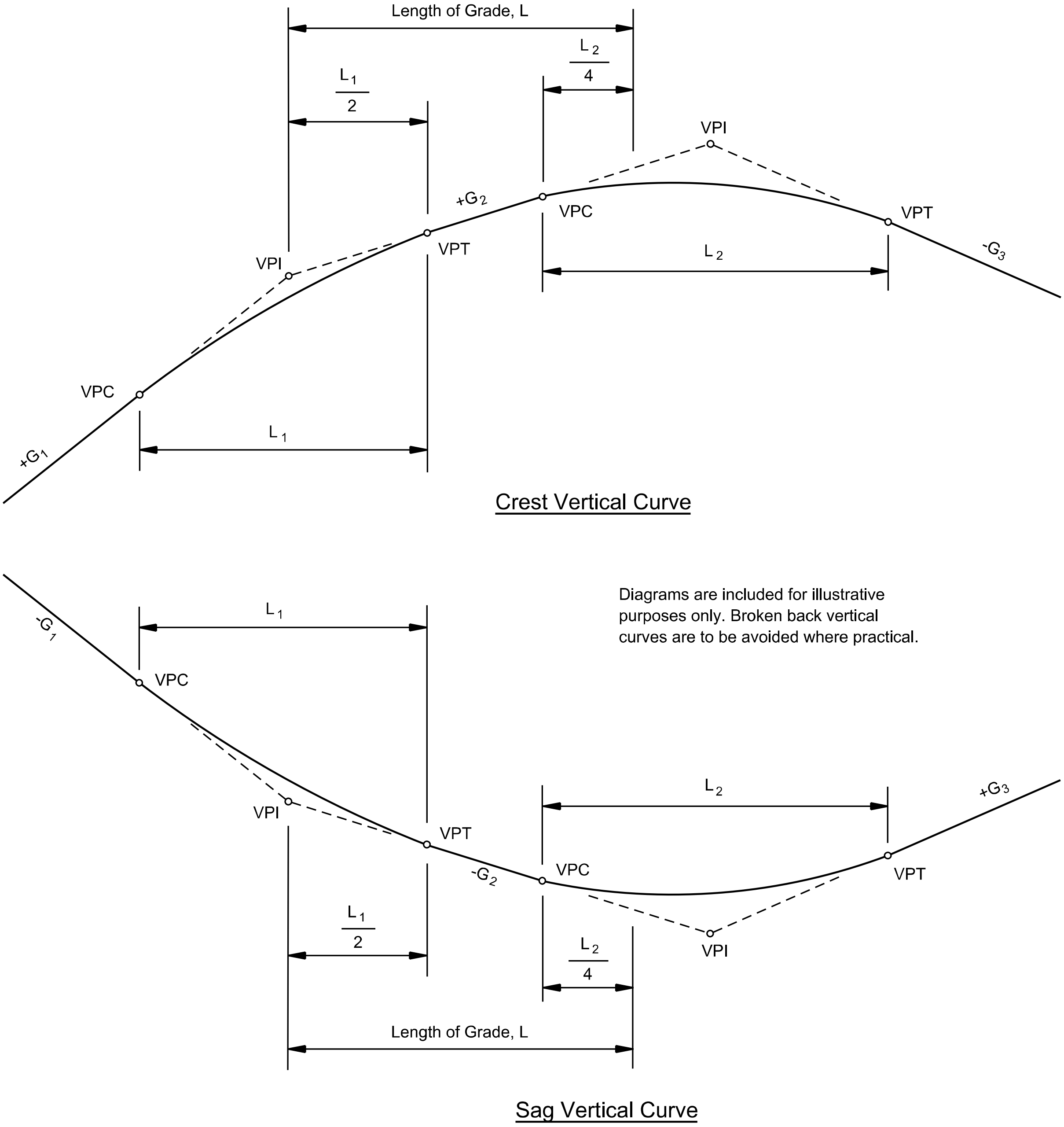
Example 6.2-1

Given: Level Approach
G = +4%
L = 450 m (length of grade)
Rural Arterial

Problem: Determine if the critical length of grade is exceeded.

Solution: Figure 6-1 yields a critical length of grade of 350 m for a 15 km/h speed reduction. The length of grade (L) exceeds this value. Therefore, either flatten the grade, if practical, or evaluate the need for a truck-climbing lane.

Figure 6-2: Measurement for Length of Grade



Notes:

1. *For vertical curves where the two tangent grades are in the same direction (both upgrades or both downgrades), 50% of the curve length will be part of the length of grade.*

2. *For vertical curves where the two tangent grades are in opposite directions (one grade up and one grade down), assume 25% of the curve length will be part of the length of grade.*

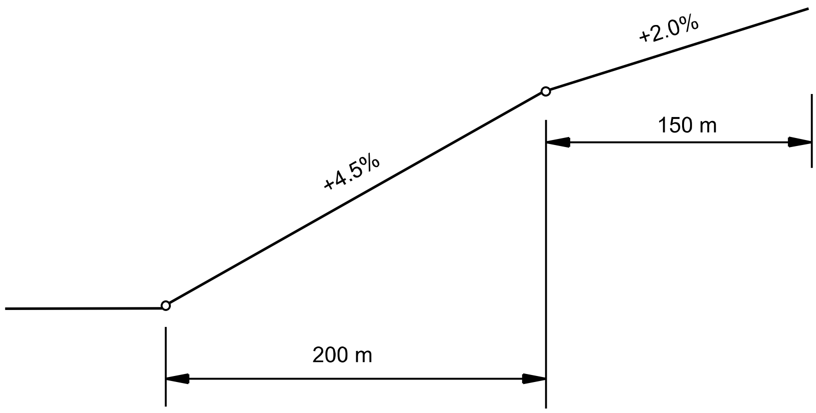
Example 6.2-2

Given: Level Approach
G_1 = +4.5%
L_1 = 200 m
G_2 = +2%
L_2 = 150 m See Figure 6-3.
Rural Arterial with a significant number of heavy trucks

Problem: Determine if the critical length of grade is exceeded for the combination of grades G_1 and G_2.

Solution: From Figure 6-1, G_1 yields a truck speed reduction of approximately 10 km/h. G_2 yields a speed reduction of approximately 2 km/h. The total of 12 km/h is less than the maximum 15 km/h speed reduction. Therefore, the critical length of grade is not exceeded.

Figure 6-3: Example 6.2-2



Example 6.2-3

Given: Figure 6-4 illustrates the vertical alignment on a low-volume, two-lane rural collector roadway with no large trucks.

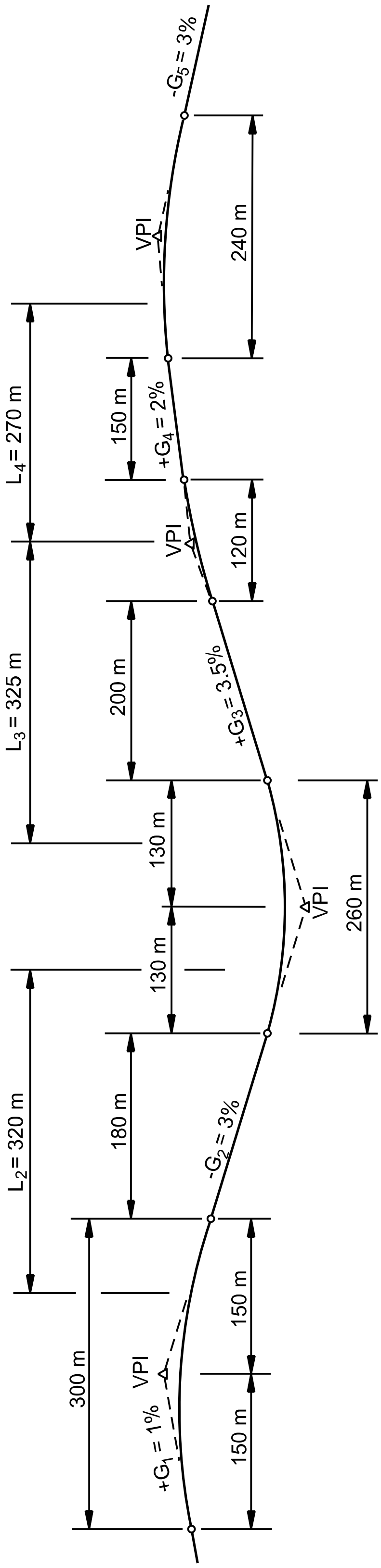
Problem: Determine if the critical length of grade is exceeded for G_2 or for the combination upgrade G_3 and G_4.

Solution: Use the following steps:

Step 1. Determine the length of grade using the criteria in Figure 6-2. For this example, the lengths are determined as follows:

$$L_2 = \frac{300}{4} + 180 + \frac{260}{4} = 320 \text{ m}$$

Figure 6-4: Critical Length of Grade Calculations (Example 6.2-3)



$$L_3 = \frac{260}{4} + 200 + \frac{120}{2} = 325 \text{ m}$$

$$L_4 = \frac{120}{2} + 150 + \frac{240}{4} = 270 \text{ m}$$

Step 2. Determine the critical length of grade for the roadway in both directions. Use Figure 6-1 to determine the critical length of grade.

a. For trucks travelling left to right, enter into Figure 6-1 the value for G_3 (3.5%) and L_3 = 320 m. The speed reduction is approximately 13 km/h. For G_4 (2%) and L_4 = 270 m, the speed reduction is approximately 5 km/h. The total speed reduction on the combination upgrade G_3 and G_4 is 18 km/h. This exceeds the maximum 15 km/h speed reduction. However, on low-volume roads, the designer can assume a 10 km/h increase in truck speed for the 3% momentum grade (G_2), which precedes G_3. Therefore, a speed reduction may be as high as 25 km/h before concluding that the combination grade exceeds the critical length of grade. Assuming the benefits of the momentum grade, this leads to the conclusion that the critical length of grade is not exceeded.

b. Next, determine the critical length of grade for trucks travelling in the opposite direction. In Figure 6-1, enter in 3% for G_2 and L_2 = 320 m. The speed reduction is 10 km/h. Because the speed reduction is less than 15 km/h, the critical length of grade for this direction is not exceeded.

6.3 Truck-Climbing Lanes

It is desirable to provide a truck-climbing lane as an added lane for the upgrade direction of a roadway where the grade, traffic volumes, and heavy-vehicle volumes combine to degrade traffic operations from those on the approach to grade. See Figure 6-5.

Figure 6-5: Truck Climbing Lane

6.3.1 Location Guidelines

A truck-climbing lane may be necessary to allow a specific upgrade to operate at an acceptable level of service. The following criteria will apply:

1. Two-Lane Roadways (Single Carriageway). On a two-lane, two-way single carriageway, provide a truck-climbing lane if the following conditions are satisfied:

   a. the upgrade traffic flow is in excess of 200 veh/h; and

   b. the heavy-vehicle volume (i.e. trucks, buses and recreational vehicles) exceeds 20 veh/h during the design hour; and

   c. one of the following conditions exists:

      - the critical length of grade is exceeded for the 15 km/h speed reduction curve (see Figure 6-1), or
      - the level of service (LOS) on the upgrade is E or F, or
      - there is a reduction of two or more LOS when moving from the approach segment to the upgrade; and

   d. the construction costs and the construction impacts (e.g. environmental, right-of-way) are considered reasonable.

2. Multilane Roadways (Dual Carriageway). Provide a truck-climbing lane on a dual carriageway if the following conditions are satisfied:

   a. the directional service volume for LOS D is exceeded on the upgrade; and

   b. the directional service volume exceeds 1000 veh/h/lane; and

   c. one of the following conditions exists:

      - the critical length of grade is exceeded for the 15 km/h speed reduction curve (see Figure 6-1), or
      - the LOS on the upgrade is E or F, or
      - there is a reduction of one or more LOS when moving from the approach segment to the upgrade; and

   d. the construction costs and the construction impacts (e.g. environmental, right-of-way) are considered reasonable.

Also, provide truck-climbing lanes where the above criteria are not met and if there is an adverse crash experience on the upgrade related to slow-moving heavy vehicles.

6.3.2 Capacity Analysis

A capacity analysis for truck-climbing lanes is determined in the same manner as that used for two-lane roadways or multilane roadways, as applicable. Chapter 8 “Rural Roads” and the *Highway Capacity Manual 2010* (17) provide guidance in determining the level of service criteria for truck-climbing lanes. The designer is required to gather the following information to conduct a capacity analysis for truck-climbing lanes:

- roadway functional classification,
- lane width,
- shoulder width,
- access-point density (one side),
- terrain,
- percent no-passing zones,
- speed limit,
- design speed,
- length of passing lane (if present),
- pavement condition,
- hourly passenger car volume,
- length of analysis period,
- peak hour factor,
- directional split,
- heavy vehicle percentage, and
- percent occupied for parking.

This data is used to determine the capacity of the truck-climbing lane or the service flow rate that can be accommodated at any given LOS.

6.3.3 Design Guidelines

Table 6-1 summarises the design criteria for a truck-climbing lane. Figure 6-6 illustrates the design criteria for a truck-climbing lane. In addition, consider the following:

1. Entering Speed. For entering speeds equal to or greater than 100 km/h, use 100 km/h for the truck design speed. For entering speeds less than 100 km/h, use the roadway design speed or the posted speed limit, whichever is less. Under restricted conditions, the designer may want to consider the effect a momentum grade will have on the entering speed. See Comment 3 in Section 6.2.5 for additional information on momentum grades. Note that the maximum speed is 100 km/h.

2. Cross Slope. On tangent sections, the truck-climbing lane cross slope will be the same as that of the adjacent travel lane.

3. Performance Curves. The effect of rate and length of grade on the speed of a typical heavy truck is shown in Figure 6-7. From Figure 6-7, it can be determined how far a 120 kg/kW truck, starting its climb from any speed up to approximately 110 km/h, travels up various grades or combinations of grades before a certain or uniform speed is reached. For instance, with an entering speed of 90 km/h, the truck travels approximately 600 m up a 6% grade before its speed is reduced to 50 km/h.

Table 6-1: Design Criteria for Truck-Climbing Lanes

Design Element	Desirable	Minimum
Lane width	4.0 m	3.65 m
Shoulder width	Same width as approach shoulder	Freeways/Expressways: 3.0 m Other Roadways: 1.2 m
Cross slope on tangent	2%	2%
Beginning of full-width lane (1)	Location where the truck speed has been reduced to 15 km/h below the posted speed limit	Location where the truck speed has been reduced to 60 km/h
End of full width lane (2)	Location where truck has reached posted speed or 90 km/h, whichever is less	Location where truck has reached a speed of 15 km/h below the posted speed
Entering taper	25:1	90 m
Exiting taper	50:1	180 m
Minimum full-width length	300 m or greater	300 m or greater

Notes:

1. *Use Figure* 6-7 *to determine truck deceleration rates. In determining the applicable truck speed, the designer may consider the effect of momentum grades.*

2. *Use Figure* 6-7 *to determine truck acceleration rates.*

Figure 6-6: Truck Climbing Lane

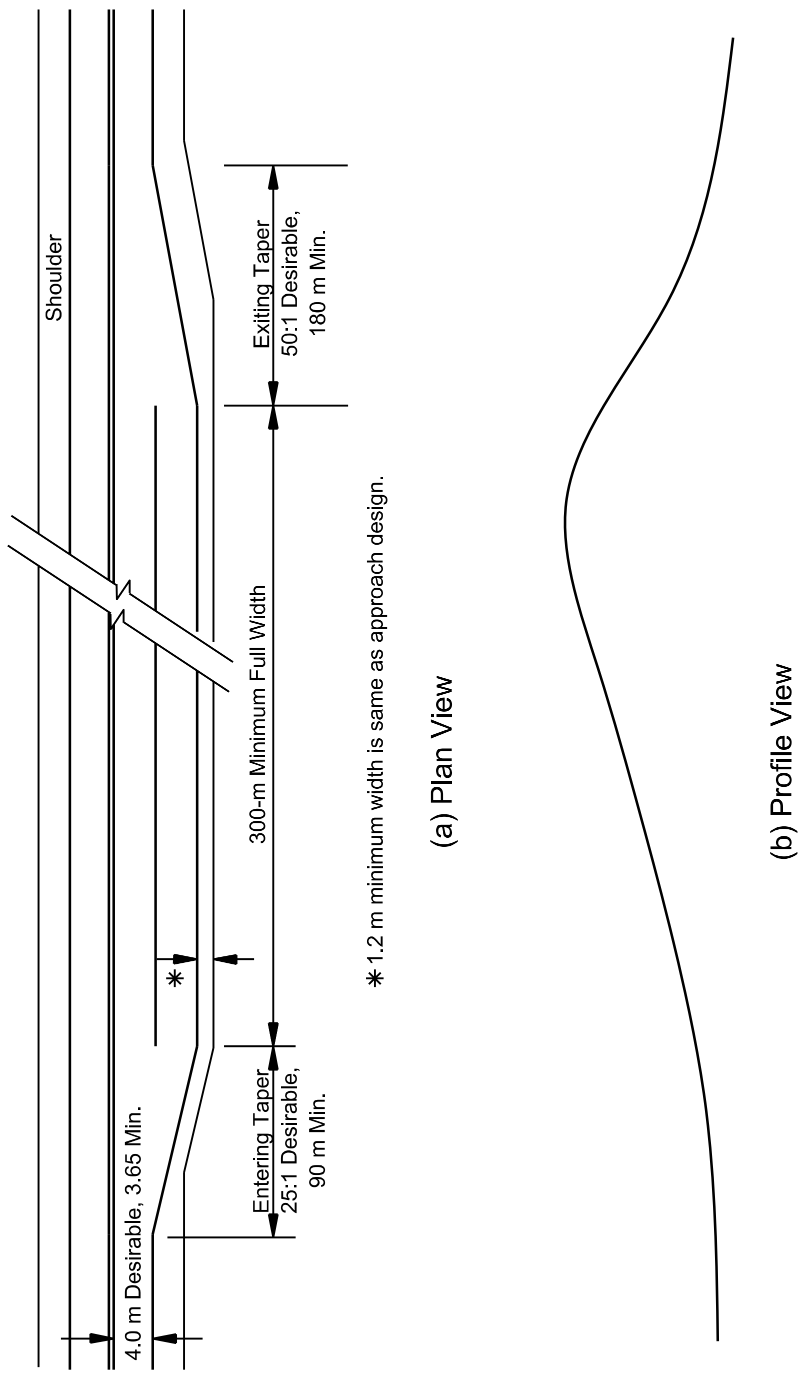
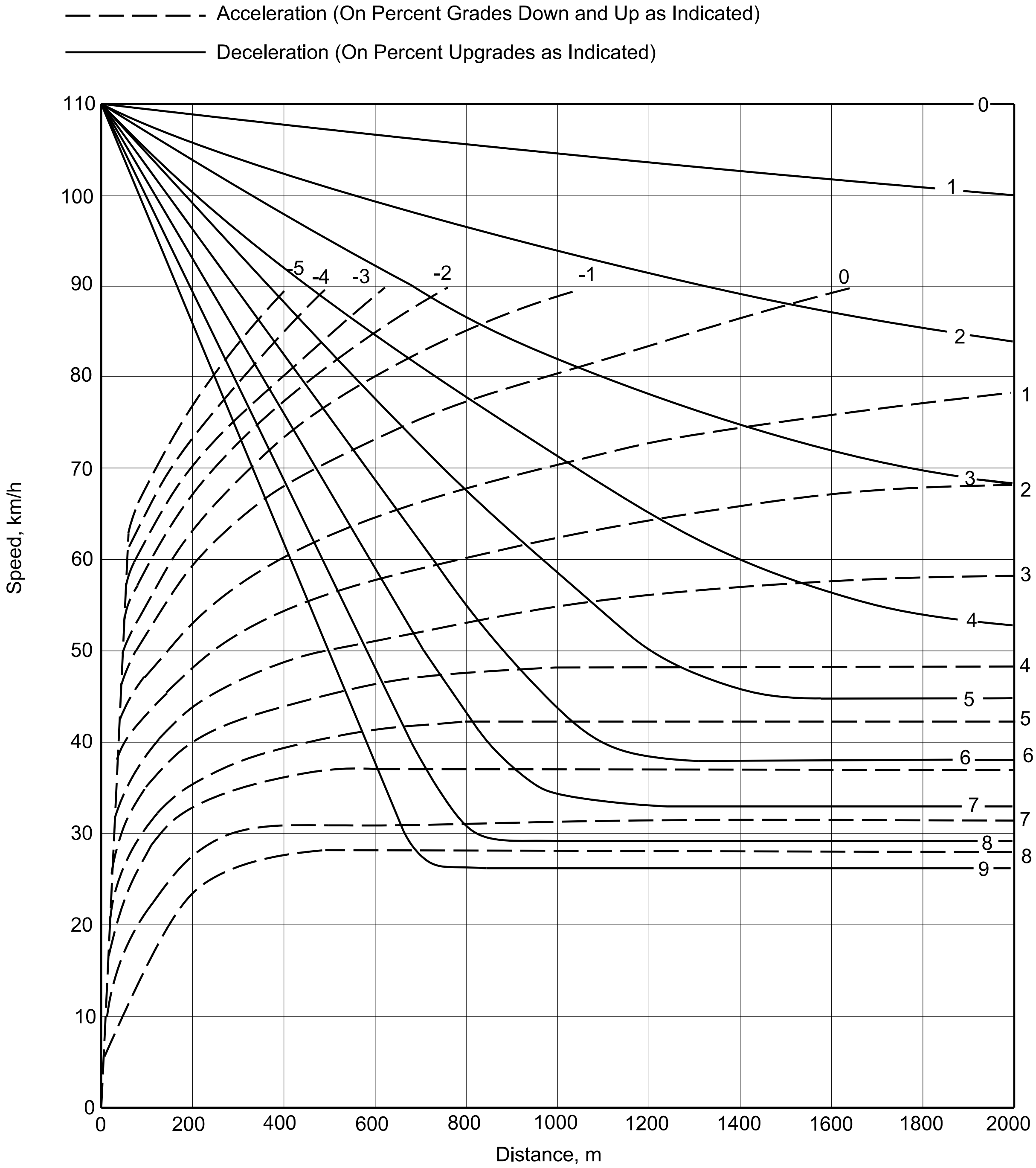


Figure 6-7: Performance Curves for a Typical Heavy Truck (120 kg/kW)



Note: *For entering speeds equal to or greater than 110 km/h, use an initial speed of 110 km/h. For speeds less than 110 km/h, use the design speed or posted speed limit as the initial speed.*

Source: (1)

Example 6.3-1

Given: Level approach
Grade = +6% for 1000 m
V = 90 km/h (truck design speed)
Rural arterial, heavy-truck route

Problem: Determine the truck speed at the end of the 1000 m upgrade.

Solution: Use the following steps:

Step 1. In Figure 6-7, determine the distance where the 90 km/h horizontal line crosses the +6% gradeline (solid line). This occurs at approximately 300 m.

Step 2. Move along the +6% grade for an addition 1000 and determine the truck's final speed at the top of the grade of approximately 38 km/h.

1. End of Full-Width Lane. In addition to the criteria in Table 6-1, ensure that there is sufficient sight distance available to the point where the truck will begin to merge back into the through travel lane. At a minimum, this will be the stopping sight distance. Desirably, the driver should have decision sight distance available to the roadway surface (i.e. height of object = 0.0 m) at the end of the taper.

   The full-lane width should be extended beyond the crest vertical curve and not ended just beyond the crest of the grade. Desirably the full-lane width should not end on a horizontal curve.

2. Signing and Pavement Markings. See the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) for guidance on signing and pavement markings for truck-climbing lanes.

6.3.4 Downgrades

Truck lanes on downgrades are not typically considered. However, steep downhill grades may also have a detrimental effect on the capacity and safety of facilities with high traffic volumes and numerous heavy trucks. Although specific criteria have not been established for these conditions, trucks descending steep downgrades in low gear may produce nearly as great an effect on operations as an equivalent upgrade. The need for a truck lane for downhill traffic will be considered on a site-by-site basis.

6.4 Vertical Curves

Vertical curves are curves that provide transitions between two sloped roadways, allowing a vehicle to negotiate the elevation rate change at a gradual rate rather than a sharp cut. The design of the curve is dependent on the intended design speed for the roadway, as well as other factors including drainage, slope, acceptable rate of change, and friction. These curves are parabolic and are assigned stationing based on a horizontal axis.

6.4.1 Crest Vertical Curves

6.4.1.1 Design Equations

The basic equations for determining the minimum length of a crest vertical curve are:

$$L = \frac{AS^2}{100\left(\sqrt{2h_1} + \sqrt{2h_2}\right)^2}$$

Equation 6.1: Basic Equation (S < L)

$$K = \frac{S^2}{200\left(\sqrt{h_1} + \sqrt{h_2}\right)^2}$$

Equation 6.2: K-Value (S < L)

$$L = KA$$

Equation 6.3: Length of Curve

where: L = length of vertical curve, m
A = algebraic difference between the two tangent grades, %
S = sight distance, m
h_1 = height of eye above road surface, m
h_2 = height of object above road surface, m
K = horizontal distance needed to produce a 1% change in gradient.

The length of a crest vertical curve will depend upon A for the specific curve and upon the selected sight distance (e.g., stopping, passing, decision), height of eye, and height of object. Equations 6.1, 6.2, and 6.3 and the resultant values of K are predicated on the sight distance being less than the length of vertical curve. However, these values can also be used, without significant error, where the sight distance is greater than the length of vertical curve. The following sections discuss the selection of K-values. For design purposes, round the calculated length (L) up to the next highest 10-m increment.

6.4.1.2 Stopping Sight Distance

The principal control in the design of crest vertical curves is to ensure that minimum stopping sight distance (SSD) is available throughout the vertical curve. Figure 6-8 (a) shows the design controls for crest vertical curves. The following discusses the application of K-values for various operational conditions:

1. Passenger Cars (Level Grade). Table 6-2 presents K-values for passenger cars on a level grade. Level conditions are assumed where the grade on the far side of the vertical curve is less than 3%. These K-values are calculated by assuming h_1 = 1.080 m, h_2 = 0.600 m, and S = SSD in the basic equations for crest vertical curves (Equations 6.1, 6.2 and 6.3). The values represent the lowest acceptable sight distance on a facility. Where cost effective, use higher stopping sight distances.

Table 6-2: K Values for Crest Vertical Curves – Stopping Sight Distance (Passenger Cars – Level Grades)

Design Speed (km/h)	Stopping[1] Sight Distance (m)	Rate of Vertical Curvature, K[2]	
		Calculated	Design
20	20	0.6	1
30	35	1.9	2
40	50	3.8	4
50	65	6.4	7
60	85	11.0	11
70	105	16.8	17
80	130	25.7	26
90	160	38.9	39
100	185	52.0	52
110	220	73.6	74
120	250	95.0	95
130	285	123.4	124
140[3]	325	161.0	161

Notes:

1. *Stopping sight distances (SSD) are from Table 4-1.*

$$K = \frac{SSD^2}{658}, \text{ where: } h_1 = 1.080 \text{ m}, h_2 = 0.600 \text{ m}$$

2. *Drainage affects design of vertical curves where curbed sections are used. The heavy line in the table above represents a drainage criterion of K=51. It is not intended for the criterion to be considered a maximum, but merely a value beyond which drainage should be more carefully evaluated.*

3. *Estimated by DMT.*

Source: (1)

2. Passenger Cars (Grade Adjusted). For a given speed, the safe stopping distance on downgrades is greater than that for a level roadway. Where grades on the far side of a crest vertical curve are –3% or greater, design the length of curve using K-values that have been adjusted for the increased braking distances resulting from the downgrade. No adjustment is necessary for grades less than 3% or for upgrades. Table 6-3 presents the K-values for passenger cars adjusted for downgrades.

3. Minimum Length. The minimum length of a crest vertical curve in metres should be 0.6V, where V is the design speed in km/h. These distances apply regardless of the calculated length of vertical curve. For aesthetics, the suggested minimum length of a crest vertical curve on a rural roadway is 300 m.

6.4.1.3 Decision Sight Distance

At some locations, decision sight distance may be warranted in the design of crest vertical curves. Section 4.4 discusses candidate sites and provides design values for decision sight distance. In complex environments, decision sight distance provides drivers with additional time to adjust their speed and additional distance to make unexpected manoeuvres. Crest vertical curve designed with decision sight distance will be longer than those using stopping sight distances. These “S” values should be used in the basic equations for crest vertical curves (Equations 6.1, 6.2, and 6.3). In addition, the following will apply:

1. Height of Eye (h_1). For passenger cars, h_1 = 1.080 m.

2. Height of Object (h_2). Decision sight distance, in many cases, is predicated upon the same principle as stopping sight distance; i.e. the driver needs sufficient distance to see a 0.600-m object. Therefore, h_2 = 0.600 m at many locations. However, at some elevations, decision sight distance may be determined assuming the pavement surface (e.g. freeway exit gores). At these locations, h_2 = 0.0 m.

3. K-Values. Table 6-4 presents the K-values for passenger cars using the decision sight distances presented in Section 4.4.

6.4.1.4 Passing Sight Distance

At some locations, it may be desirable to provide passing sight distance in the design of crest vertical curves. Section 4.5 discusses the application and design values for passing sight distances on two-lane, two-way carriageways. These “S” values are used in the basic equations for crest vertical curves (Equations 6.1, 6.2, and 6.3). In addition, the following will apply:

1. Height of Eye (h_1). For passenger cars, h_1 = 1.080 m.

2. Height of Object (h_2). Passing sight distance is predicated upon the passing driver being able to see a sufficient portion of the top of the oncoming car. Therefore, h_2 = 1.080 m.

3. K-Values. Table 6-5 presents the K-values for passenger cars using the passing sight distances presented in Section 4.5.

Table 6-3: K-Values for Crest Vertical Curves – Stopping Sight Distance (Passenger Cars – Adjustment for Downgrades)

Design Speed (km/h)	(3%)	(4%)	(5%)	(6%)	(7%)	(8%)	(9%)	(10%)
20	1	1	1	1	1	1	1	1
30	2	2	2	2	2	2	2	2
40	4	4	4	4	4	4	4	4
50	7	7	7	7	8	8	9	9
60	12	12	12	13	13	14	14	15
70	18	19	20	20	22	23	23	25
80	28	29	30	32	33	35	36	38
90	41	42	44	46	48	51	53	56
100	57	60	63	65	68	72	76	80
110	78	82	86	90	94	100	105	111
120	105	110	115	121	127	134	141	150
130	138	144	151	159	167	175	186	198
140	178	185	195	205	216	228	242	257

Notes:

1. *K-values in table have been determined by using the SSD rounded for design from Table 4-2.*
2. *For grades less than 3%, no adjustment is necessary; i.e. use the level K-values in Table 6-2.*
3. *For grades intermediate between table values, use a straight-line interpolation in Table 6-3 or use Equation 4.1 and roundup to the next highest 1-m increment to determine the SSD and then calculate the appropriate K-value.*

$$K = \frac{SSD^2}{658}, \text{ where: } h_1 = 1.080\ m, h_2 = 0.600\ m$$

4. *Drainage affects design of vertical curves where curbed sections are used. The heavy line in the table above represents a drainage criterion of K=51. It is not intended for the criterion to be considered a maximum, but merely a value beyond which drainage should be more carefully evaluated.*

Table 6-4: K-Values for Crest Vertical Curves — Decision Sight Distance (Passenger Cars)

Design Speed (km/h)	Avoidance Manoeuvre				
	A	**B**	**C**	**D**	**E**
50	7	37	32	44	58
60	14	58	44	64	84
70	20	84	61	84	115
80	30	119	80	111	151
90	44	161	111	151	197
100	61	208	151	192	243
110	84	268	166	219	281
120	107	336	197	262	336
130	141	419	231	308	395
140	181	511	243	336	451

Notes:

1. *See Section 4.4 for decision sight distances (DSD).*

$$K = \frac{DSD^2}{658}, \text{ where: } h_1 = 1.080\ m, h_2 = 0.600\ m$$

2. *Where it is desirable to see the road surface, the object height, h_2, may be set at zero and the K-values recalculated.*

Table 6-5: K-Values for Crest Vertical Curves – Passing Sight Distance (Passenger Cars)

Design Speed (km/h)	Passing(1) Sight Distance (m)	K-Value(2)
50	160	30
60	180	38
70	210	52
80	245	70
90	280	91
100	320	119
110	355	146
120	395	181
130	440	224
140	490	277

Notes:

1. *Design passing sight distances (PSD) are from Section 4.5.*

$$K = \frac{PSD^2}{864}, \text{ where: } h_1 = 1.080\ m, h_2 = 1.080\ m$$

2. *Drainage affects design of vertical curves where curbed sections are used. The heavy line in the table above represents a drainage criterion of K=51. It is not intended for the criterion to be considered a maximum, but merely a value beyond which drainage should be more carefully evaluated.*

Source: (1)

6.4.1.5 Drainage

Proper drainage must be considered in the design of crest vertical curves on curbed sections, bridges, and medians with concrete barriers. Typically, drainage problems will not be experienced if the vertical curvature is sharp enough so that a minimum longitudinal gradient of at least 0.3% is reached at a point about 15 m from either side of the apex. To ensure that this objective is achieved, determine the length of the crest vertical curve assuming a K-value of 51 or less. Where a crest vertical curve lies within a curbed section or bridge and where the maximum drainage K-value is exceeded, carefully evaluate the drainage design near the apex. See the Abu Dhabi *Road Drainage Manual* (26) for more information.

For uncurbed sections of highway, drainage should not be a problem at crest vertical curves. However, it is still desirable to provide a longitudinal gradient of at least 0.15% at points about 15 m on either side of the high point. To achieve this, K must equal 100 or less.

6.4.2 Sag Vertical Curves

Sag vertical curves are in the shape of a parabola. Typically, they are designed to allow the vehicular headlights to illuminate the roadway surface (i.e. the height of object = 0.0 m) for a given distance “S.” The light beam from the headlights is assumed to have a 1 degree upward divergence from the longitudinal axis of the vehicle. These assumptions yield the following basic equations for determining the minimum length of sag vertical curves:

$$\mathbf{L = \frac{AS^2}{200\left[h_3 + S(\tan 1^\circ)\right]} = \frac{AS^2}{200h_3 + 3.5S}}$$

Equation 6.4: Basic Sag Vertical Curve Equation (S < L)

$$\mathbf{K = \frac{S^2}{200h_3 + 3.5S}}$$

Equation 6.5: K-Value (S < L)

$$\mathbf{L = KA}$$

Equation 6.6: Length of Curve

where:	L	=	length of vertical curve, m
	A	=	algebraic difference between the two tangent grades, %
	S	=	sight distance, m
	h_3	=	height of headlights above pavement surface, m
	K	=	horizontal distance needed to produce a 1% change in gradient

The length of a sag vertical curve will depend upon (A) for the specific curve and upon the selected sight distance and headlight height. Equations 6.4, 6.5, and 6.6 and the resultant values of (K) are predicated on the sight distance being less than the length of vertical curve. However, these values can also be used, without significant error, where the sight distance is greater than the length of vertical curve. The following sections discuss the selection of K-values for sag vertical curves.

6.4.2.1 Stopping Sight Distance

The principal control in the design of sag vertical curves is to ensure a minimum stopping sight distance (SSD) is available for headlight illumination throughout the sag vertical curve. Figure 6-8 (b) shows the design controls for sag vertical curves. The following discusses the application of K-values for various operational conditions:

1. Passenger Cars (Level Grade). Table 6-6 presents K-values for passenger cars. These are calculated by assuming h_3 = 0.600 m and S = SSD in the basic equations for sag vertical curves (Equations 6.4, 6.5, and 6.6). The minimum values represent the lowest acceptable sight distance on a facility. However, because sag vertical curves greatly affect the aesthetics of a roadway alignment, use longer than the minimum lengths of curves to provide a more aesthetically pleasing design.

2. Passenger Cars (Grade Adjusted). For a given speed, the safe stopping sight distance on downgrades is greater than that for a level surface. For sag vertical curves, only consider grade adjustments when the sag curve is between two downgrades and where the downgrades are -3% or greater. Table 6-7 presents K-values adjusted for grades for passenger cars.

3. Minimum Length. The minimum length of a sag vertical curve, in metres, should be 0.6V, where V is the design speed in km/h. For aesthetics on rural roadways, the minimum length of a sag vertical curve is dependent upon the driver’s view of the roadway. The greater the distance a roadway can be seen ahead, the longer the sag vertical curve should be.

One exception to the minimum length on sag vertical curves applies in curbed sections and on bridges. If the sag is in a low point, the use of the minimum length criteria may produce longitudinal slopes too flat to drain storm water without ponding. For additional guidance, see Abu Dhabi *Road Drainage Manual* (26).

6.4.2.2 Decision Sight Distance

At some locations, decision sight distance may be warranted in the design of sag vertical curves. Section 4.4 discusses candidate sites and provides design values for decision sight distance. In complex environments, decision sight distance provides drivers with additional time to adjust their speed or additional distance to make unexpected manoeuvres. Sag vertical curves designed with decision sight distance will be longer than those using stopping sight distances. These “S” values should be used in the basic equations for sag vertical curves (Equations 6.4, 6.5, and 6.6). The height of headlights h_3 = 0.600 m. Table 6-8 provides K-values for sag vertical curves using decision sight distance.

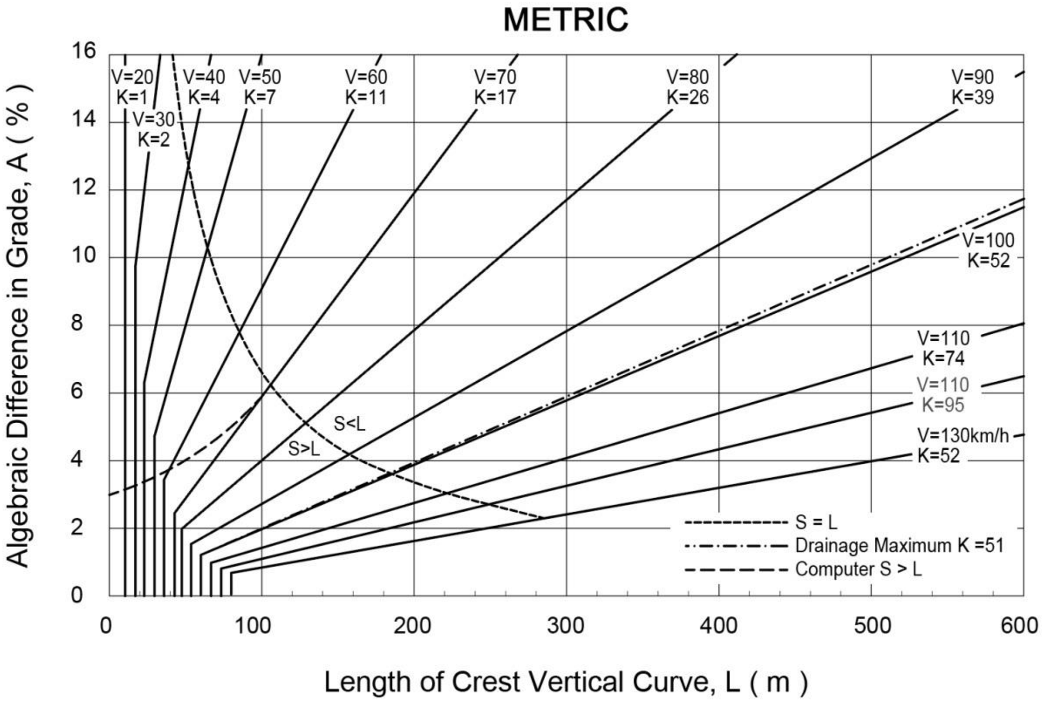


Figure 6-8 : Design Controls for Crest Vertical Curves, for Stopping Sight Distance - Upper Range.

from AASHTO, “A Policy on Geometric Design of Highways and Streets”, 2011

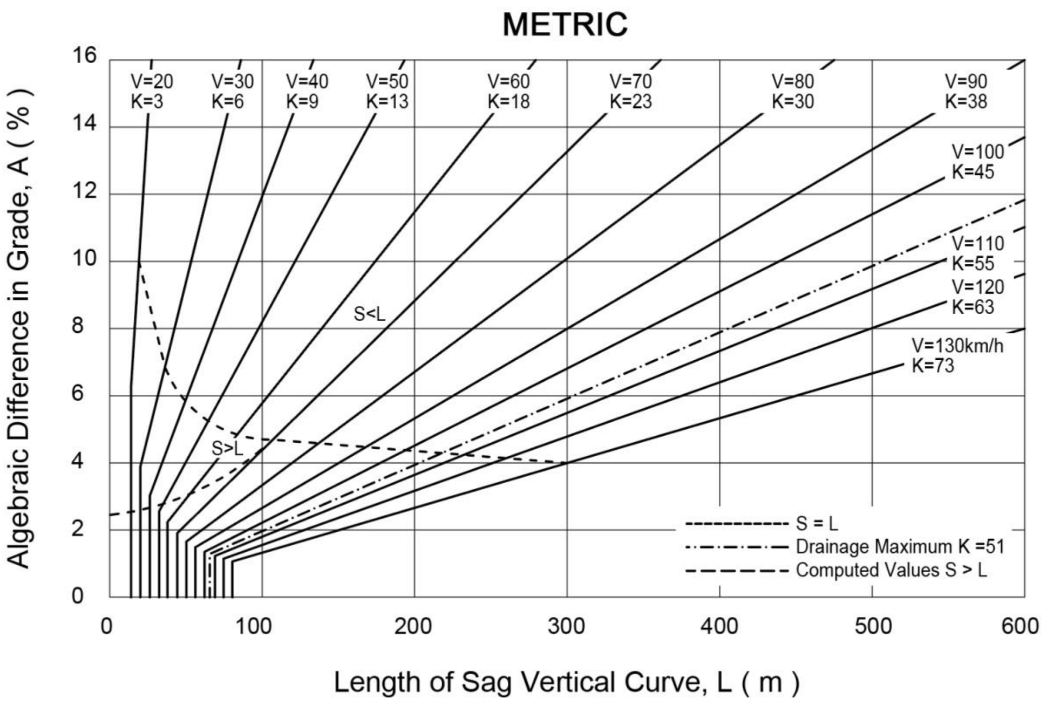


Figure 6-9: Design Controls for Sag Vertical Curves - Upper Range.

from AASHTO, “A Policy on Geometric Design of Highways and Streets”, 2011

Table 6-6: K-Values for Sag Vertical Curves – Stopping Sight Distance (Passenger Cars – Level Grades)

Design Speed (km/h)	Stopping Sight Distance (m)	Rate of Vertical Curvature, K	
		Calculated (m)	Design (m)
20	20	2.1	3
30	35	5.1	6
40	50	8.5	9
50	65	12.2	13
60	85	17.3	18
70	105	22.6	23
80	130	29.4	30
90	160	37.6	38
100	185	44.6	45
110	220	54.4	55
120	250	62.8	63
130	285	72.7	73
140[3]	325	84.0	84

Notes:

1. *Stopping sight distances (SSD) are from Table 4-1.*

$$K = \frac{SSD^2}{120 + 3.5\ SSD}, \text{ where: } h_3 = 0.600\ m$$

2. *Drainage affects design of vertical curves where curbed sections are used. The heavy line in the table above represents a drainage criterion of K = 51. It is not intended for the criterion to be considered a maximum, but merely a value beyond which drainage should be more carefully evaluated.*
3. *Estimated by DMT.*

Source: (1)

Table 6-7: K-Values for Sag Vertical Curves- Stopping Sight Distance (Passenger Cars – Adjusted For Downgrades)

Design speed (km/h)	(3%)	(4%)	(5%)	(6%)	(7%)	(8%)	(9%)	(10%)
20	2	2	2	2	2	2	2	2
30	4	5	5	5	5	5	5	5
40	8	8	8	8	9	9	9	9
50	12	13	13	13	14	14	14	15
60	18	18	19	19	19	20	20	21
70	24	25	25	26	26	27	28	29
80	31	32	32	33	34	35	36	37
90	39	40	41	42	43	44	45	47
100	47	48	50	51	52	54	55	57
110	56	58	59	61	63	65	66	68
120	66	68	70	72	74	76	78	81
130	77	79	81	83	86	88	91	94
140	89	91	93	96	99	102	105	108

Notes:

1. *K-values in table have been determined by using the SSD rounded for design from Table 4-2.*
2. *For grades less than 3%, no adjustment is necessary; i.e. use the level K-values in Table 6-6.*
3. *For grades intermediate between table values, use a straight-line interpolation in Table 4-2 to determine the SSD and then calculate the appropriate K-value.*

   $$K = \frac{SSD^2}{120 + 3.5\ SSD}, \text{ where: } h_3 = 0.600\ m$$

4. *Drainage affects design of vertical curves where curbed sections are used. The heavy line in the table above represents a drainage criterion of K = 51. It is not intended for the criterion to be considered a maximum, but merely a value beyond which drainage should be more carefully evaluated.*

Table 6-8: K-Values for Sag Vertical Curves – Decision Sight Distance (Passenger Cars)

Design Speed (km/h)	Avoidance Manoeuvre				
	A	B	C	D	E
50	13	36	34	40	47
60	20	47	40	50	59
70	25	59	49	59	70
80	32	71	57	68	81
90	40	84	68	81	94
100	49	97	81	92	105
110	59	111	85	100	114
120	67	125	94	110	125
130	78	141	102	119	137
140	90	156	105	125	146

Notes:

1. *See Section 4.4 for discussion on decision sight distances (DSD).*
2. $K = \frac{DSD^2}{120 + 3.5\ DSD}$, *where:* $h_3 = 0.600\ m$

6.4.2.3 Comfort Criteria

On fully lighted, continuous sections of roadway and where it is impractical to provide stopping sight distance, a sag vertical curve may be designed to meet the comfort criteria. These criteria are based on the effect of change in the vertical direction of a sag vertical curve due to the combined gravitational and centrifugal forces. The consensus is that riding is comfortable on sag vertical curves when the centripetal acceleration does not exceed 0.3 m/s^2. The length-of-curve equation for the comfort criteria is:

$$L = \frac{AV^2}{395}$$

Equation 6.7:

where: L = length of vertical curve, m
A = algebraic difference between the two tangent grades, %
V = design speed, km/h

6.4.2.4 Underpass

Check sag vertical curves through underpasses to ensure that the underpass structure does not obstruct the driver's visibility. Use the

Equation 6.8 to check sag vertical curves through underpasses where the sight distance is less than the length of curve (S < L).

$$L = \frac{AS^2}{800(C - 1.5)}$$

Equation 6.8: Underpass (S < L)

Use Equation 6.9 to check sag vertical curves through underpasses where the sight distance is greater than the length of curve (S>L).

$$L = 2S - \frac{800\,(C - 1.5)}{A}$$

Equation 6.9: Underpass (S > L)

where:

where:	L	=	length of vertical curve, m
	A	=	algebraic difference between the two tangent grades, %
	S	=	sight distance, m
	C	=	vertical clearance of underpass, m

Compare the L calculated from

Equation 6.8 for underpasses with the L calculated based on headlight illumination (Equation 6.4). The larger of the two lengths will govern.

6.4.2.5 Drainage

Proper drainage must be considered in the design of sag vertical curves on curbed sections, bridges, and medians with concrete barriers. Drainage problems are minimised if the sag vertical curve is sharp enough so that both of the following criteria are met:

- a minimum longitudinal gradient of at least 0.3% is reached at a point about 15 m from either side of the low point, and
- there is at least a 100-mm elevation differential between the low point in the sag and the two points 15 m to either side of the low point.

To ensure that the first objective is achieved, base the length of the vertical curve upon a K-value of 51 or less. For most design speeds, the K-values are less than 51; see Table 6-6. However, for higher design speeds and/or where longer sag vertical curves are required in curbed sections or on bridges, it may be necessary to install flanking inlets on either side of the low point.

For uncurbed sections of highway, drainage should not be a problem at sag vertical curves.

6.4.3 Vertical Curve Computations

The following will apply to the mathematical design of vertical curves:

1. Definitions. Table 6-9 presents the common terms and definitions used in vertical curve computations.

2. Symmetrical Vertical Curve. All measurements for vertical curves are made on the horizontal or vertical plane, not along the profile gradeline. With the simple parabolic curve, the vertical offsets from the tangent vary as the square of the horizontal distance from the VPC or VPT. Elevations along the curve are calculated as proportions of the vertical offset at the point of vertical intersection (VPI). The necessary equations for computing a symmetrical vertical curve are shown in Figure 6-10.

3. Unsymmetrical Vertical Curve. Occasionally, it is necessary to use an unsymmetrical vertical curve to obtain clearance on a structure or to meet other existing field conditions. This curve is similar to the parabolic vertical curve, except the curve does not vary symmetrically about the VPI. Note that, with the unsymmetrical vertical curve, the curve is treated as two separate parabolas. The necessary equations for computing an unsymmetrical vertical curve are shown in Figure 6-12.

6.5 Minimum Vertical Clearances

The following are minimum vertical clearance requirements for roadway structures, pedestrian overpasses, railroad overpasses, tunnels and sign structures. Lesser clearances may be used only under very restrictive conditions, upon individual analysis and with prior the approval from the roadway authority during the project concept stage.

1. Roadway Structures, Pedestrian Overpasses, Sign Structures. The minimum design vertical clearance for roadways passing under roadway structures, pedestrian overpasses, and sign structures shall be at least 6.50 m (6.00 m in urban areas) over the entire roadway width, including auxiliary lanes and shoulders. Add an allowance for the possible long-term deflection and future resurfacing.

2. Channel Crossings. For bridges over channels, the minimum overhead clearance within navigable shipping channels shall be greater than 8.5 m above the high-tide water level. The navigable width shall be as required by the applicable authorities.

3. Railroad Overpasses. Structures over all railways shall provide a minimum clearance of 7.5 m from the top of rail. Additional allowance may need to be provided for future track adjustments.

4. Tunnels and Underpasses. The minimum design vertical clearance for roadways within tunnels shall be at least 6.5 m (6.00 m in urban areas). Add an allowance for future

resurfacing. A minimum vertical clearance of 3.5 m is required for pedestrian tunnels and 6.5 m for camel underpasses.

5. Detours. For detour routes, a temporary 6.0 m minimum vertical clearance will be acceptable.

6.6 Alignment and Profile Relationships

Figure 6-13 illustrates a typical plan and profile sheet.

Table 6-9: Vertical Curve Definitions

Element	Abbreviation	Definition
Vertical Point of Curvature	VPC	The point at which a tangent grade ends and the vertical curve begins.
Vertical Point of Tangency	VPT	The point at which the vertical curve ends and the tangent grade begins.
Vertical Point of Intersection	VPI	The point where the extension of two tangent grades intersect.
Grade	G_1,G_2	The rate of slope between two adjacent VPIs expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in metres for each 100 m of horizontal distance. Upgrades in the direction of stationing are identified as plus (+). Downgrades are identified as minus (–).
External Distance	E	The vertical distance (offset) between the VPI and the roadway surface along the vertical curve.
Algebraic Difference in Grade	A	The value of A is determined by the deflection in percent between two tangent grades ($G_2 – G_1$).
Length of Vertical Curve	L	The horizontal distance in metres from the VPC to the VPT.
Tangent Elevation	Tan. Elev.	The elevation on the tangent line between the VPC and VPI and the VPI and VPT.
Elevation on Vertical Curve	Curve Elev.	The elevation of the vertical curve at any given point along the curve.
Horizontal Distance	x	Horizontal distance measured from the VPC or VPT to any point on the vertical curve, in metres.
Tangent Offset	y	Vertical distance from the tangent line to any point on the vertical curve, in metres.
Low/High Point	x_T	The station at the high point for crest curves or the low point for sag curves. At this point, the slope of the tangent to the curve is equal to 0%.
Symmetrical Curve	—	The VPI is located at the mid-point between VPC and VPT stationing
Unsymmetrical Curve	—	The VPI is not located at the mid-point between VPC and VPT stationing.

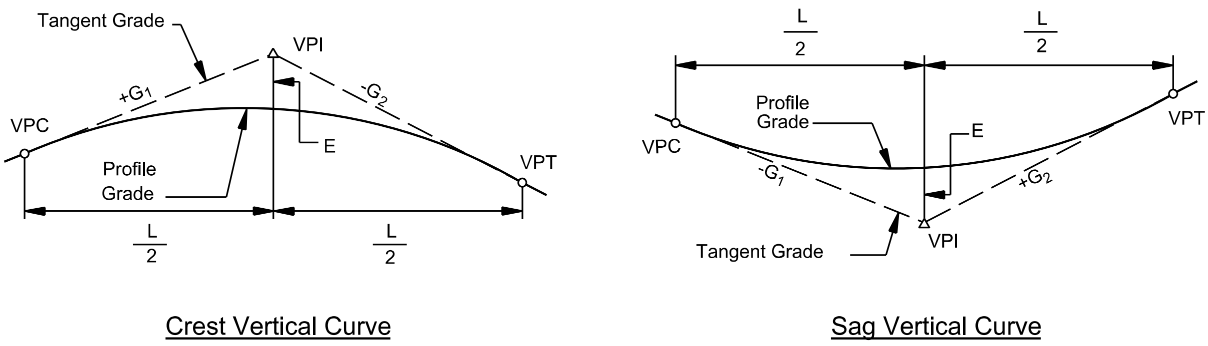


Figure 6-10: Symmetrical Vertical Curve Equation

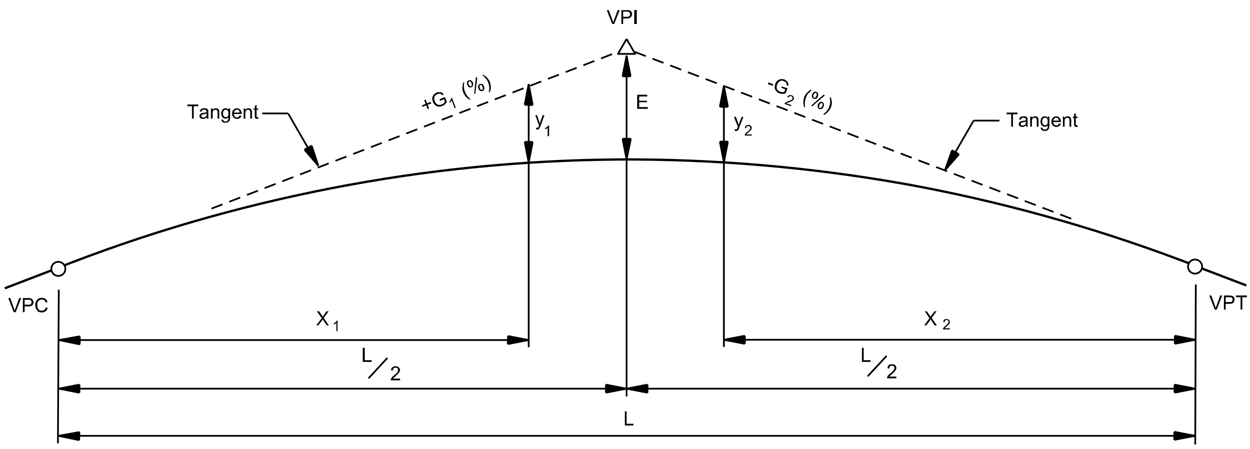


Figure 6-11: Symmetrical Vertical Curve Equations (*Continued*)

E = External distance @VPI, m
y = Any tangent offset, m
L = Horizontal length of vertical curve, m
x = Horizontal distance from VPC or VPT to any ordinate “y”, m
G_1 & G_2 = Rates of grade, expressed algebraically, %

NOTE: ALL EXPRESSIONS TO BE CALCULATED ALGEBRAICALLY
(Use algebraic signs of grades; grades in percent.)

1. <u>Elevations of VPC and VPI</u>:

$$\text{VPC ELEV.} = \text{VPI ELEV.} - \left(\frac{G_1}{100} \times \frac{L}{2}\right)$$ **Equation 6.10:**

$$\text{VPT ELEV.} = \text{VPI.} + \left(\frac{G_2}{100} \times \frac{L}{2}\right)$$ **Equation 6.11:**

2. <u>For the elevation of any point "x" on a vertical curve</u>:

$$\text{CURVE ELEV.} = \text{TAN ELEV.} \pm y$$ **Equation 6.12:**

Where:

Left of VPI (x_1 measured from VPC):

(a) $$\text{TAN ELEV.} = \text{VPC ELEV.} + \left(\frac{G_1}{100}\right)x_1$$ **Equation 6.13:**

(b) $$y_1 = x_1^2 \frac{(G_2 - G_1)}{200\ L}$$ **Equation 6.14:**

Right of VPI (x2 measured from VPT):

(a) $$\text{TAN ELEV.} = \text{VPT ELEV.} - \left(\frac{G_2}{100}\right)x_2$$ **Equation 6.15:**

(b) $$y_2 = {x_2}^2 \frac{(G_2 - G_1)}{200\ L}$$ **Equation 6.16:**

At the VPI:

$y = E$ and $x = L/2$

(a) $$\text{TAN ELEV.} = \text{VPC ELEV.} + \frac{G_1 L}{200}$$

or $$\text{TAN ELEV.} = \text{VPT ELEV.} - \frac{G_2 L}{200}$$ **Equation 6.17:**

(b) $$E = \frac{L(G_2 - G_1)}{800}$$ **Equation 6.18:**

3. Calculating high or low point in the vertical curve:

(a) To determine distance "x_T" from VPC:

$$x_T = \frac{L\ G_1}{G_2 - G_1}$$ **Equation 6.19:**

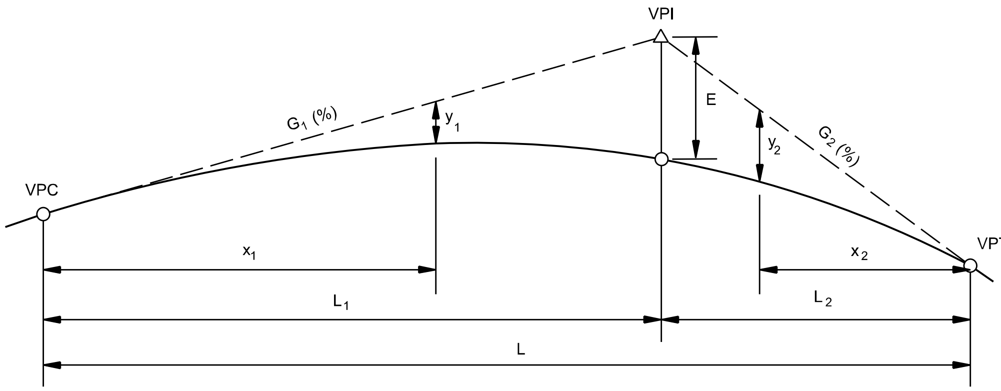
(b) To determine high or low point stationing:

VPC Sta. + x_T **Equation 6.20:**

(c) To determine high or low point elevation on a vertical curve:

$$\text{ELEV.}_{\text{HIGH OR LOW POINT}} = \text{VPC ELEV.} - \frac{L\,G_1^2}{(G_2 - G_1)\,200}$$ **Equation 6.21:**

Figure 6-12: Unsymmetrical Vertical Curve Equations



E = Offset from the VPI to the curve (external distance), m
y = Any tangent offset, m
L = Horizontal length of vertical curve, m
L_1 = Horizontal distance from VPC to VPI, m
L_2 = Horizontal distance from VPT to VPI, m
x = Horizontal distance from VPC or VPT to any ordinate "y", m
G_1 & G_2 = Rates of grade, expressed algebraically, %

Note: ALL EXPRESSIONS TO BE CALCULATED ALGEBRAICALLY
(Use algebraic signs of grades; grades in percent.)

1. Elevations of VPC and VPI:

$$\text{VPT ELEV.} = \text{VPI ELEV.} + \left(\frac{G_2}{100}\right) L_2$$ **Equation 6.22:**

$$\text{VPC ELEV.} = \text{VPI ELEV.} - \left(\frac{G_1}{100}\right) L_1$$ **Equation 6.23:**

2. For the elevation of any point "x" on a vertical curve:

CURVE ELEV. = TAN. ELEV. ± y **Equation 6.24:**

Where:

Left of VPI (x_1 measured from VPC):

(a) $$\text{TAN ELEV.} = \text{VPC ELEV.} + \left(\frac{G_1}{100}\right) x_1$$ **Equation 6.25:**

(b) $$y_1 = x_1^2 \left(\frac{L_2}{L_1}\right)\left(\frac{G_2 - G_1}{200\ L}\right)$$ **Equation 6.26:**

Right of VPI (x_2 measured from VPT):

(a) $$\text{TAN ELEV.} = \text{VPT ELEV.} - \left(\frac{G_2}{100}\right) x_2$$ **Equation 6.27:**

(b) $$y_2 = x_2^2 \left(\frac{L_1}{L_2}\right)\left(\frac{G_2 - G_1}{200\ L}\right)$$ **Equation 6.28:**

At the VPI:

$y = E$ and $x = L_1$

(a) $\text{TAN ELEV.} = \text{VPC ELEV.} + \left(\frac{G_1}{100}\right) L_1$ or **Equation 6.29:**

$\text{TAN ELEV.} = \text{VPT ELEV.} - \left(\frac{G_2}{100}\right) L_2$

(b) $E = L_1 L_2 \left(\frac{G_2 - G_1}{200\ L}\right)$ **Equation 6.30:**

3. Calculating High or Low Point on a Curve:

Note: Two answers will be determined by solving the equations below. Only one answer is correct. The incorrect answer is where $x_T > L_1$ on the left side of the VPI or where $x_T > L_2$ on the right side of the VPI.

a. Assume high or low point occurs left of VPI to determine the distance, x_T, from VPC:

$$x_T = \left(\frac{L_1}{L_2}\right)\left(\frac{G_2 L}{G_1 - G_2}\right)$$ **Equation 6.31:**

Note: Does $x_T > L_1$? If yes, this answer is incorrect and the high or low point is on the right side of the VPI. (Go to Step d. to solve for the high or low point elevation.) If no, then this is the correct answer and proceed with Steps b. and c. below.)

b. To determine high or low point stationing (where $x_T < L_1$):

$\text{STA}_{\text{HIGH OR LOW POINT}} = \text{VPC STA.} + x_T$ **Equation 6.32:**

c. To determine high or low point elevation on vertical curve (when $x_T < L_1$):

$\text{ELEV.}_{\text{HIGH OR LOW POINT}} = \text{VPC ELEV.} - \left(\frac{L_1}{L_2}\right)\left(\frac{L\,G_1^2}{(G_2 - G_1)200}\right)$ **Equation 6.33:**

d. If $X_T > L_1$ from Step a., the high or low point occurs right of the VPI. Determine the distance x_T from the VPT:

$x_T = \left(\frac{L_2}{L_1}\right)\left(\frac{G_2 L}{(G_2 - G_1)}\right)$ **Equation 6.34:**

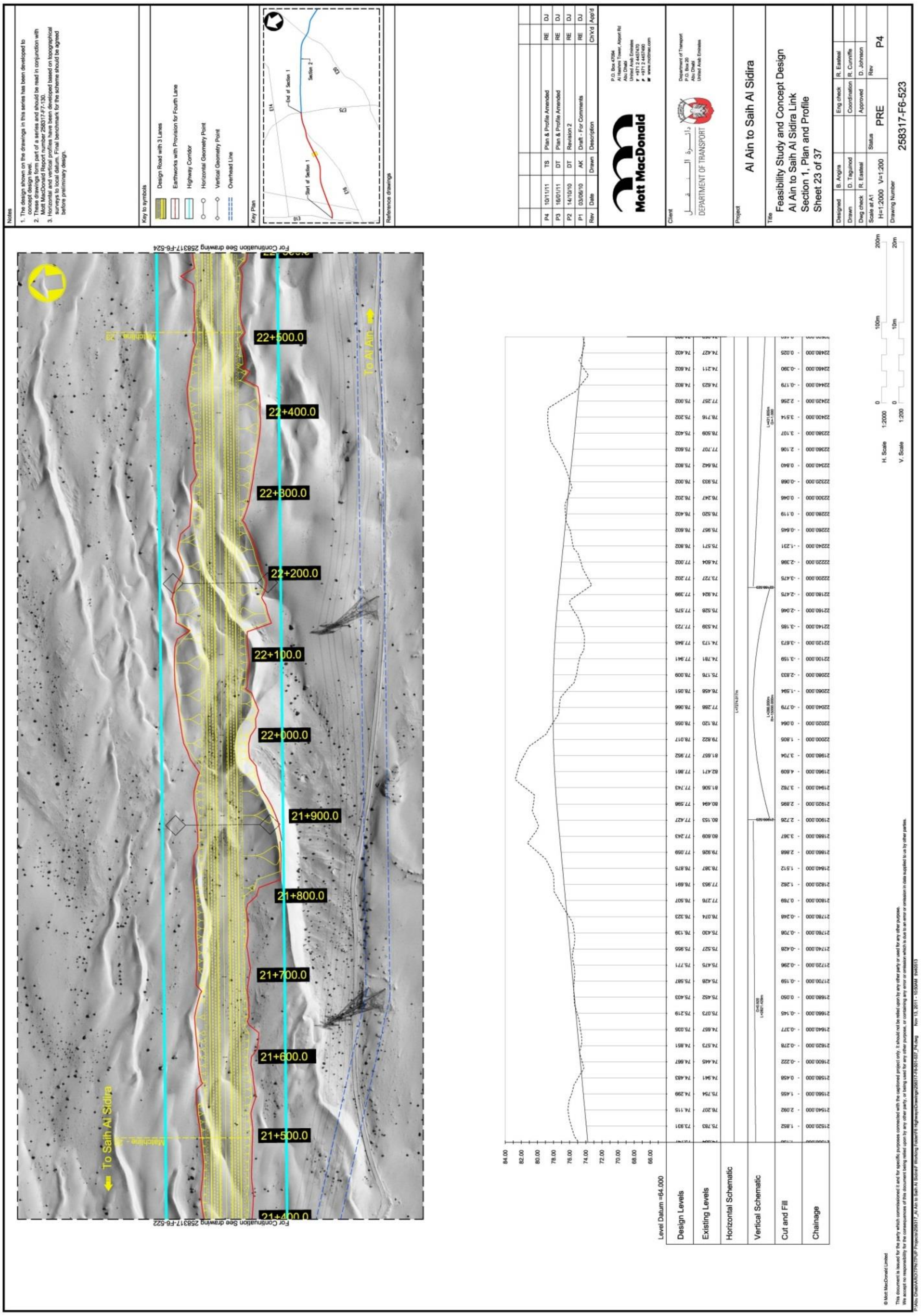
e. To determine high or low point stationing:

$\text{STA}_{\text{HIGH OR LOW POINT}} = \text{VPT STA.} - x_T$ **Equation 6.35:**

f. To determine high or low point elevation on a vertical curve:

$\text{ELEV.}_{\text{HIGH OR LOW POINT}} = \text{VPT ELEV.} - \left(\frac{L_2}{L_1}\right)\left(\frac{L\,G_2^2}{(G_2 - G_1)200}\right)$ **Equation 6.36:**

Figure 6-13: Sample Plan and Profile Sheet



6.6.1 General Controls for Vertical Alignment

The design of vertical alignment involves, to a large extent, complying with specific limiting criteria. These include maximum and minimum grades, sight distance at vertical curves and vertical clearances. In addition, the designer should adhere to certain general design principles and controls that will determine the overall safety and operation of the facility and will enhance the aesthetic appearance of the roadway. These design principles for vertical alignment include:

1. <u>Consistency</u>. Use a smooth gradeline with gradual changes, consistent with the type of roadway and character of terrain, rather than a line with numerous breaks and short lengths of tangent grades.

2. <u>Coordination with Natural/Man-Made Features</u>. The vertical alignment should be properly coordinated with the natural topography, available right-of-way, utilities, roadside development, and natural/man-made drainage patterns. This is especially important in rugged terrain.

3. <u>Roller Coaster</u>. Avoid a roller-coaster type of profile, especially where the horizontal alignment is relatively straight. This type of profile may be proposed in the interest of economy, but it is aesthetically undesirable. To avoid this type of profile, incorporate into the design horizontal curvature and/or flatter grades that may require greater excavations and higher embankments.

4. <u>Broken-Back Curvature</u>. Avoid broken-back gradelines (two crest or sag vertical curves separated by a short tangent). This alignment is particularly noticeable on divided roadways with open-ditch median sections. One long vertical curve is more desirable.

5. <u>Long Grades</u>. On a long ascending grade, it is preferable to place the steepest grade at the bottom and flatten the grade near the top. It is also preferable to break the sustained grade with short intervals of flatter grades. Evaluate substantial lengths of grades for their effect on traffic operations (e.g. trucks).

6. <u>Sags</u>. Avoid sag vertical curves in cuts unless adequate drainage can be provided. Also, to avoid drainage problems on bridges, do not place the low point of sag vertical curves on a bridge.

7. <u>Intersections</u>. Maintain moderate grades through intersections to facilitate braking and turning movements.

8. <u>Environmental Impacts</u>. Vertical alignment should be properly coordinated with environmental impacts (e.g. encroachment onto wetlands).

9. <u>Sand Control</u>. Ensure the profile is 1.5 m to 2.0 m above the surrounding terrain.

6.6.2 Coordination of Horizontal and Vertical Alignment

Do not design the horizontal and vertical alignments independently. Instead, they should complement each other. This is especially true for new construction projects. Poorly coordinated designs can detract from the benefits and emphasise the deficiency of each alignment. Their importance demands that the designer carefully evaluate the interdependence of the two roadway design features.

Horizontal alignments and vertical profiles are among the most important permanent design elements for a roadway. Excellence in their design and coordination increases the roadway's utility and safety, encourages uniform speeds, and can greatly improve the roadway's appearance. This usually can be accomplished with little additional costs. The designer should coordinate the layout of the horizontal and vertical alignment as early as practical in the design process. Alignment layouts are typically completed after the topography and ground line have been drafted.

It is difficult to discuss the combination of horizontal alignment and vertical profile without reference to the broader subject of roadway location. The subjects are mutually interrelated and what may be said about one generally is applicable to the other. The physical controls or influences that act singularly or in combination that determine the type of alignment are:

- the character of roadway, justified by traffic volumes;
- topography and subsurface conditions;
- existing roadway and cultural developments;
- likely future developments; and
- suitable locations for intersections and interchanges.

In addition, the designer should consider the following when coordinating horizontal and vertical alignment on rural roadways:

1. <u>Balance</u>. Horizontal curvature and grades should be in proper balance. Maximum curvature with flat grades or flat curvature with maximum grades does not achieve this desired balance. A compromise between the two extremes produces the best design relative to safety, capacity, ease, uniformity of operations, and aesthetics.

2. <u>Coordination</u>. Vertical curvature superimposed upon horizontal curvature (i.e. vertical and horizontal PIs at approximately the same stations) generally results in a more pleasing appearance and reduces the number of sight distance restrictions. Successive changes in profile not in combination with horizontal curvature may result in a series of humps visible to the driver for some distance, which may produce an unattractive design. However, under some circumstances, superimposing the horizontal and vertical alignment must be tempered somewhat by Comments 3 and 4 below.

3. <u>Crest Vertical Curves</u>. Do not introduce sharp horizontal curvature at or near the top of pronounced crest vertical curves. This is undesirable because the driver cannot perceive the horizontal change in alignment, especially at night when headlight beams project straight ahead into space. This potential hazard can be avoided if the horizontal curvature leads the vertical curvature or by using design values that well exceed the minimums.

4. <u>Sag Vertical Curves</u>. Do not introduce sharp horizontal curves at or near the low point of pronounced sag vertical curves or at the bottom of steep grades. Because visibility to

the road ahead is foreshortened, only flat horizontal curvature will avoid an undesirable, distorted appearance. At the bottom of long grades, vehicular speeds often are higher, particularly for trucks, and erratic operations may occur, especially at night.

5. Passing Sight Distance. In some cases, the need for frequent passing opportunities and a higher percentage of passing sight distance may supersede the desirability of combining horizontal and vertical alignment. In these cases, it may be necessary to provide long tangent sections to secure sufficient passing sight distance.

6. Intersections. At intersections, horizontal and vertical alignment should be as flat as practical to provide a design that produces sufficient sight distance and gradients for vehicles to slow, stop, or turn.

7. Dual Carriageways. On divided facilities with wide medians, it is frequently advantageous to provide independent alignments for dual carriageways.

8. Residential Areas. For roadways near subdivisions, design the alignment and profile to minimise nuisance factors to neighbourhoods. For freeways, a depressed facility can make the roadway less visible and reduce the noise to adjacent residents. For all roadway types, minor adjustments to the horizontal alignment may increase the buffer zone between the roadway and residential areas.

9. Aesthetics. The alignment should be designed to enhance attractive scenic views of natural and man-made elements (e.g. rivers, rock formations, parks, large bridges). The roadway should:

- head into, rather than away from special views;
- fall towards features at low elevations; and
- rise towards features that are best seen from below or silhouette against the sky.

6.6.3 Alignment Coordination Figures

Figure 6-14 through Figure 6-18 illustrate poor and preferred examples of horizontal and vertical alignment coordination.

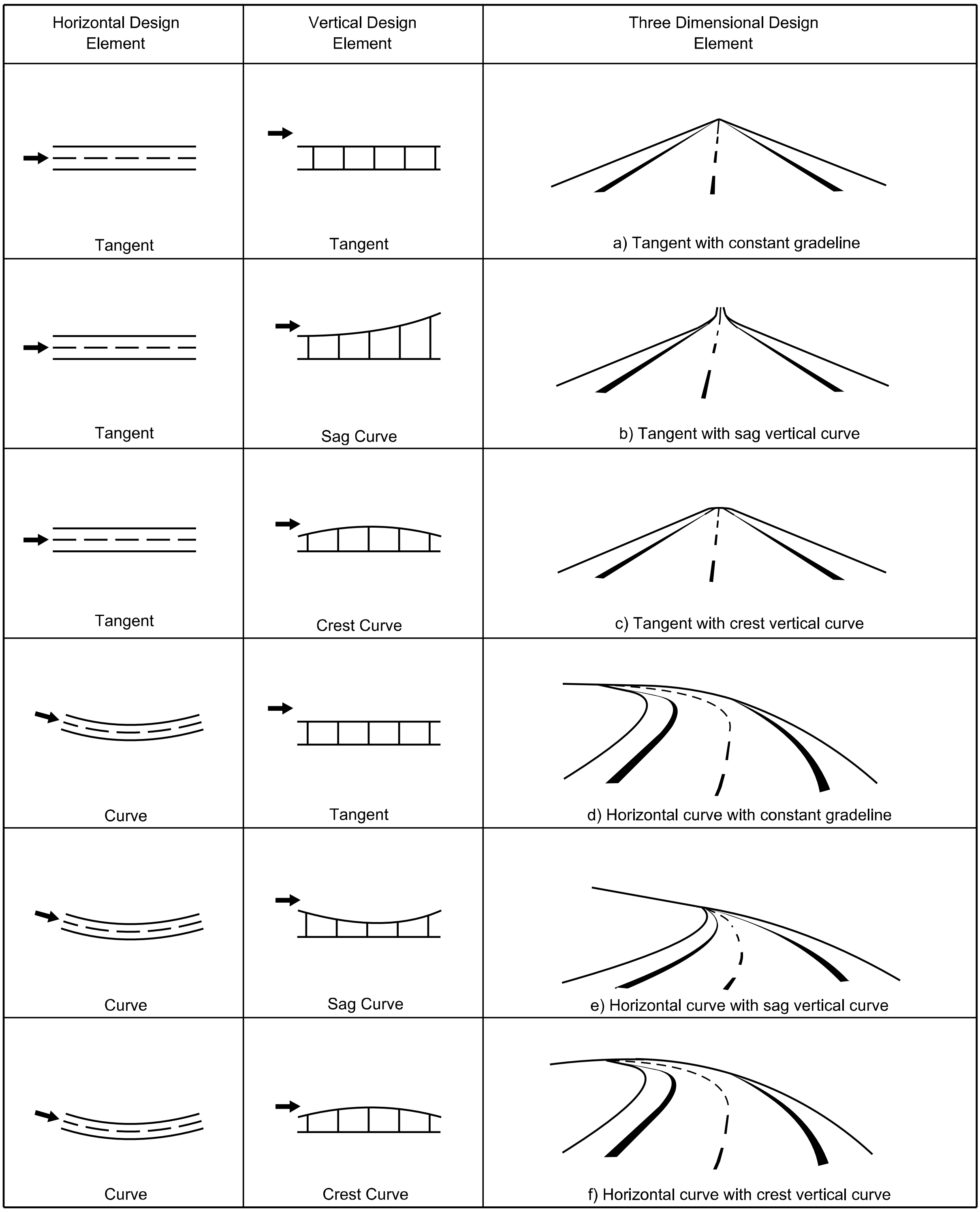
1. Good Alignment Coordination. The illustrations in Figure 6-14 show preferred combinations for coordinating horizontal and vertical alignments.

2. Tangent with Constant Gradeline. Figure 6-14(a) has constant straight horizontal and flat vertical curve alignment. This is the easiest alignment to design; however, designer needs to consider drainage requirements and that long-straight roads may cause driver fatigue in featureless terrain.

3. Tangent with Sag Vertical Curve. Figure 6-14(b) is sag vertical curve on a straight horizontal alignment. Long sag vertical curves allow the driver to see the roadway ahead and provide pleasing aesthetics.

4. Tangent with Crest Vertical Curve. Figure 6-14(c) illustrates a long crest vertical curve on a straight horizontal alignment. This design may also be used to enhance attractive scenic views of the natural and manmade environment (e.g. rivers, rock formation, parks, outstanding man-made structures).

Figure 6-14: Horizontal and Vertical Alignment Coordination



a. Horizontal Curve with Constant Gradeline. Figure 6-14(d) shows a horizontal curve in flat vertical plane. This is a good design as the driver can see the entire horizontal curve. However, avoid the use of minimum curve radii s to eliminate the appearance kink alignment.

b. Horizontal Curve with Sag Vertical Curve. Figure 6-14(e) illustrates a horizontal curve within a sag vertical curve. Ensure steep vertical declines are not used prior to the beginning of the horizontal curve. This alignment may require careful design to avoid a disjointed or kink appearance.

c. Horizontal Curve with Crest Vertical Curve. Figure 6-14(f) illustrates a horizontal curve on a crest vertical curve for this configuration. The designer should ensure that a greater than minimum radius is used for horizontal curve and that the horizontal curve starts prior to the beginning of the crest vertical curve.

5. Broken-Back Alignments. Figure 6-15 illustrates several broken-back alignments (short tangent between two curves in the same direction), which are poor designs. Where practical, replace the broken-back alignment with long horizontal and/or vertical curves to improve the design and aesthetics of the roadway.

6. Use of Minimum Curves. Figure 6-16 illustrate the resulting poor aesthetics when using minimum horizontal and/or vertical curvatures in open environments where the driver can see the roadway far into the distance. Larger than minimum values will improve the design and aesthetics of the roadway.

a. Visual Break Caused by Horizontal Tangent and Short Sag Vertical Curve. Figure 6-16(a) illustrates how the use of a short sag vertical curve in an open environment appears to cause a kink in the road.

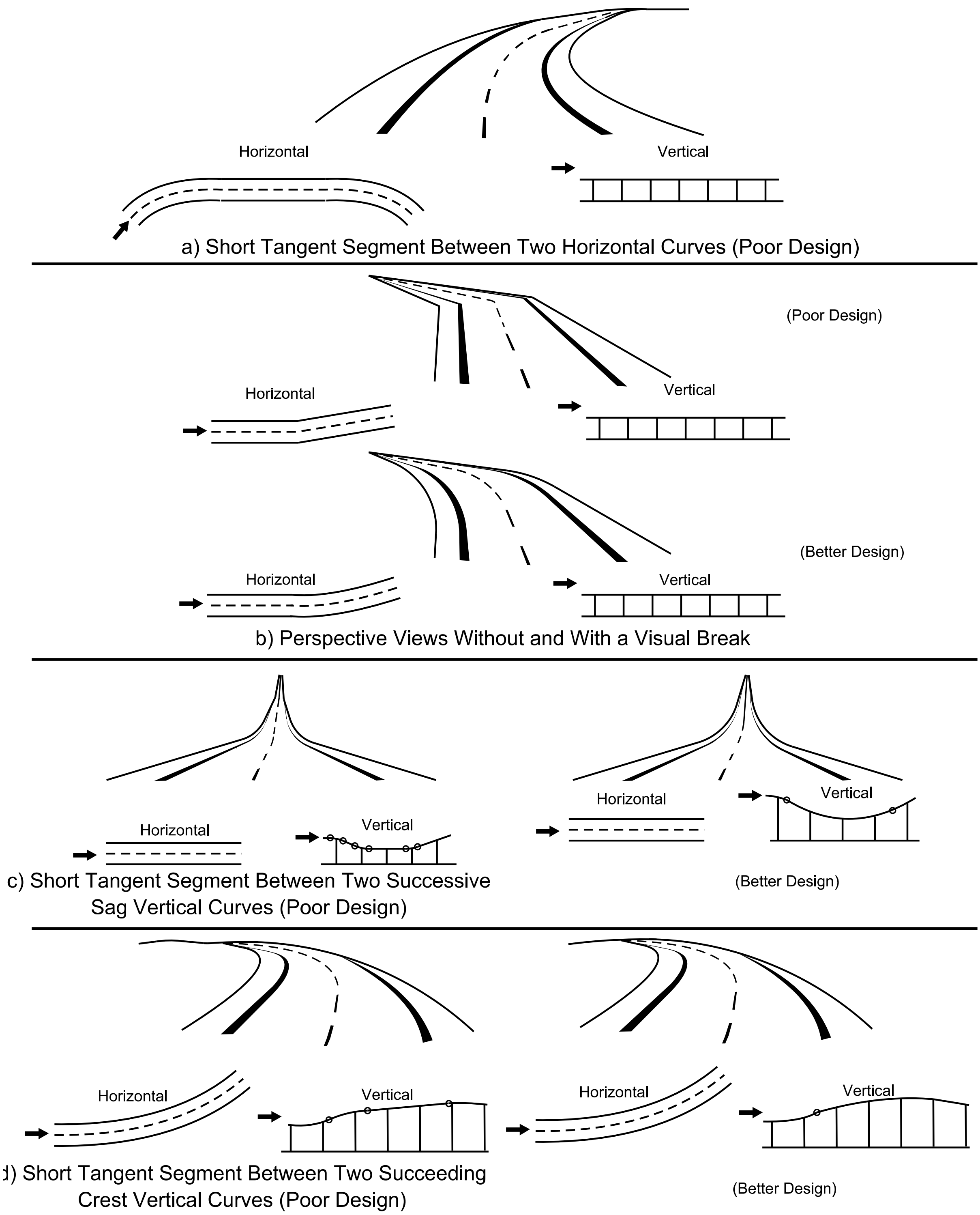
b. Visual Break Caused by Horizontal Curve and Short Sag Vertical Curve. In Figure 6-16(b), the driver's view of roadway ahead disappears and then reappears due the use of a short sag vertical curve on a long horizontal curve.

c. Visual Break with Short Vertical Curves. Figure 6-16(c) illustrates how a short crest vertical curve on a long upgrade appears to make a kink in the vertical alignment.

d. Multiple Short Vertical Curves on Tangent. Figure 6-16(d) illustrates how the use of several short vertical curves causes the roadway to disappear from the driver's view. Vertical alignment should use a smooth gradeline with gradual changes, consistent with the type of highway and character of terrain.

e. Multiple Short Vertical Curves with a Horizontal Curve. Figure 6-16(e) illustrates how the use of minimum vertical curves on a long horizontal curve makes the roadway ahead appear to be disjointed.

Figure 6-15: Examples of Undesirable and Good Alignment Coordination



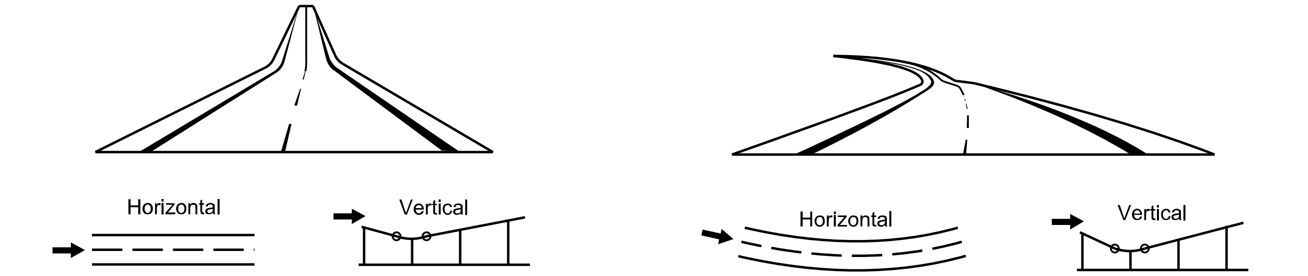
a) Short Tangent Segment Between Two Horizontal Curves (Poor Design)

b) Perspective Views Without and With a Visual Break

c) Short Tangent Segment Between Two Successive Sag Vertical Curves (Poor Design)

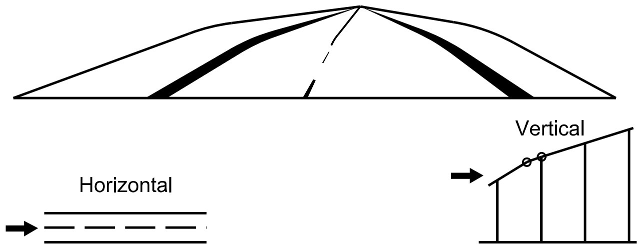
d) Short Tangent Segment Between Two Succeeding Crest Vertical Curves (Poor Design)

Figure 6-16: Combined Alignment Designs to Avoid

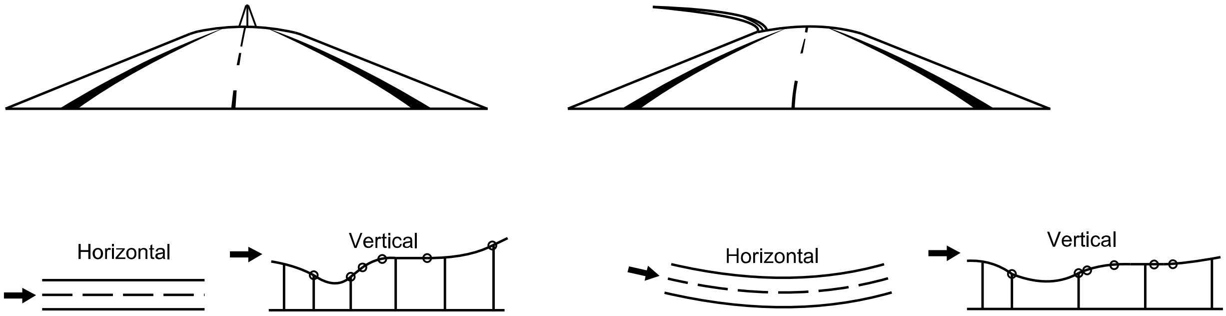


a) Visual Break Caused by Horizontal Tangent and Short Sag Vertical Curve

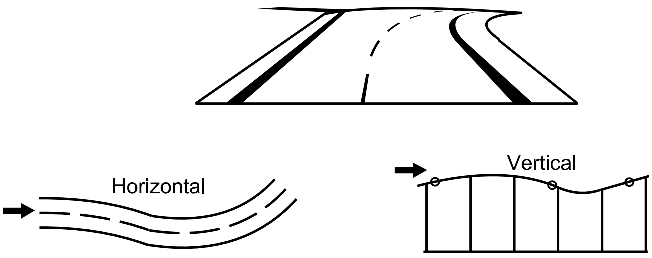
b) Visual Break Caused by Horizontal Curve and Short Sag Vertical Curve



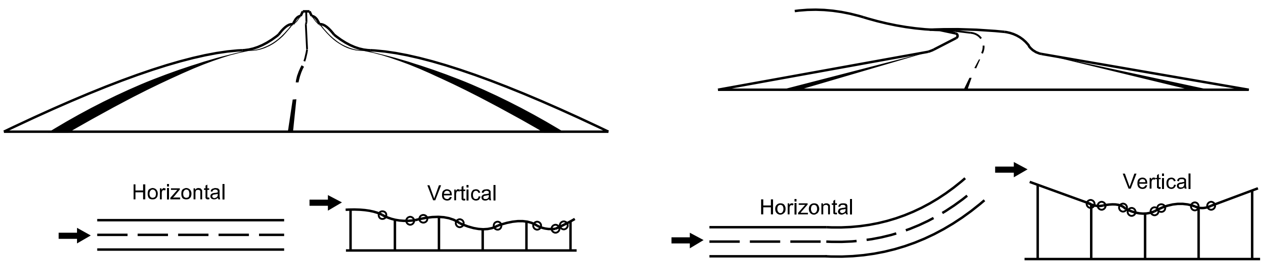
c) Visual Break with Short Crest Vertical Curve



Dip in Roadway on Tangent (d) and with a Horizontal Curve (e)



f) Rolling Profile Without Visual Restrictions on Inside of Horizontal Curve



Rollercoaster Profile in Tangent (g) and on Curve (h)

f. Short Horizontal Radii with Vertical Curves. In Figure 6-16(f), the change in the horizontal alignment is hidden due to the crest vertical curve. On high-speed facilities, this may cause undesirable operations. Desirably, the reverse horizontal alignment should be replaced with a one long horizontal curve. If this is impractical, increase the length of the crest vertical so the reverse curvature is visible to the driver.

g. Rollercoaster Profiles. Figure 6-16(g) and Figure 6-16(h) illustrate the how use of a series of short crest and sag vertical curves causes a rollercoaster alignment. This type of profile is aesthetically undesirable and may be more difficult to drive.

7. Superimposition of Horizontal and Vertical Alignment. Figure 6-17 provides guidance on superimposition of horizontal and vertical alignment.

   a. Horizontal Alignments (Curvilinear). In Figure 6-17(a), the use of short radii is acceptable because the driver is expecting the changing alignment. In the bottom illustration, the use of short radii is a poor design. The driver will not be expecting the sharp horizontal alignment after travelling a significant distance on the tangent alignment.

   b. Vertical Alignments (Relation R_c:R_s). Figure 6-17(b) show the proper placement of sag and crest vertical curve in relationship to length of the crest vertical curve. Note that the relation $R_c < R_s$ is a better design because the vertical curves are flatter.

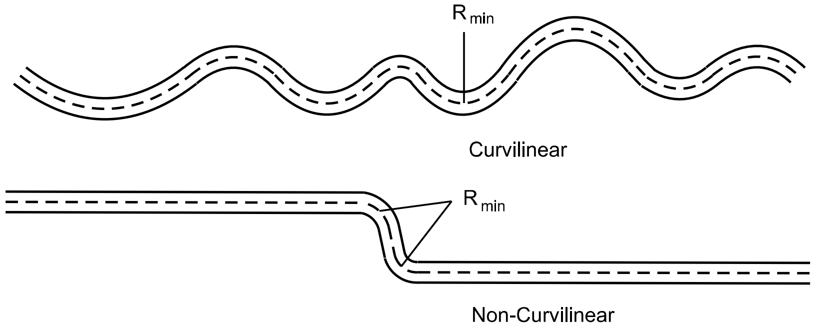
   c. Coordination of Distortion Points in Horizontal and Vertical Alignments. Figure 6-17(c) illustrates good coordination between the horizontal and vertical alignments. The driver is able to see changing horizontal alignment prior to the crest or low point of the vertical curve. Note that the “better coordination” figure uses longer and flatter vertical curves.

8. Plan and Profile Coordination. Figure 6-18 illustrates the proper coordination between the horizontal and vertical alignment.

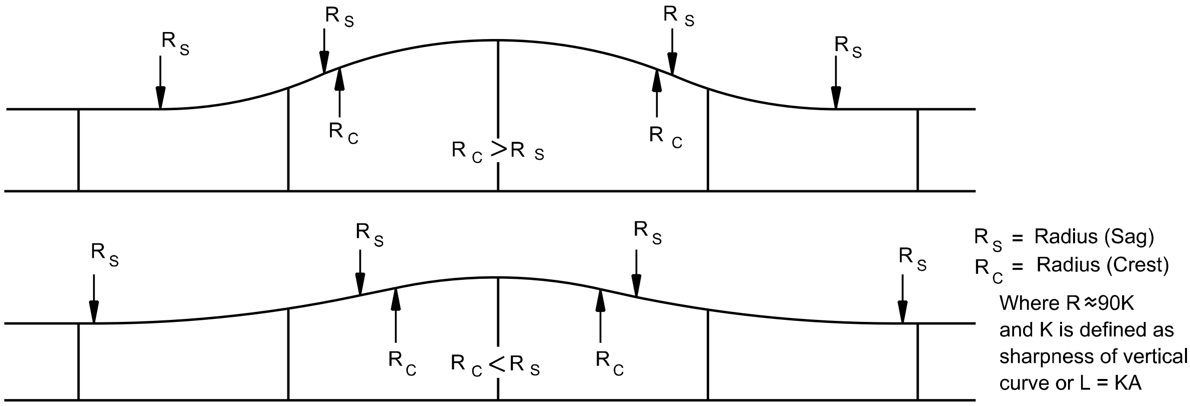
   a. Vertical and Horizontal Curve Alignment. In Figure 6-18(a), note that good designs are where the PIs for the vertical and horizontal curves coincide. The poor design is where the vertical and horizontal curves do not coincide.

   b. Break Points. Figure 6-18(b) illustrates the maximum number of break points (changes in alignment) the driver should encounter in the horizontal and vertical planes. In the horizontal plane, the driver should only be able to view two break points; and in the vertical plane, three break points. The breaks for the horizontal alignment view should also align with the breaks for the vertical alignment view.

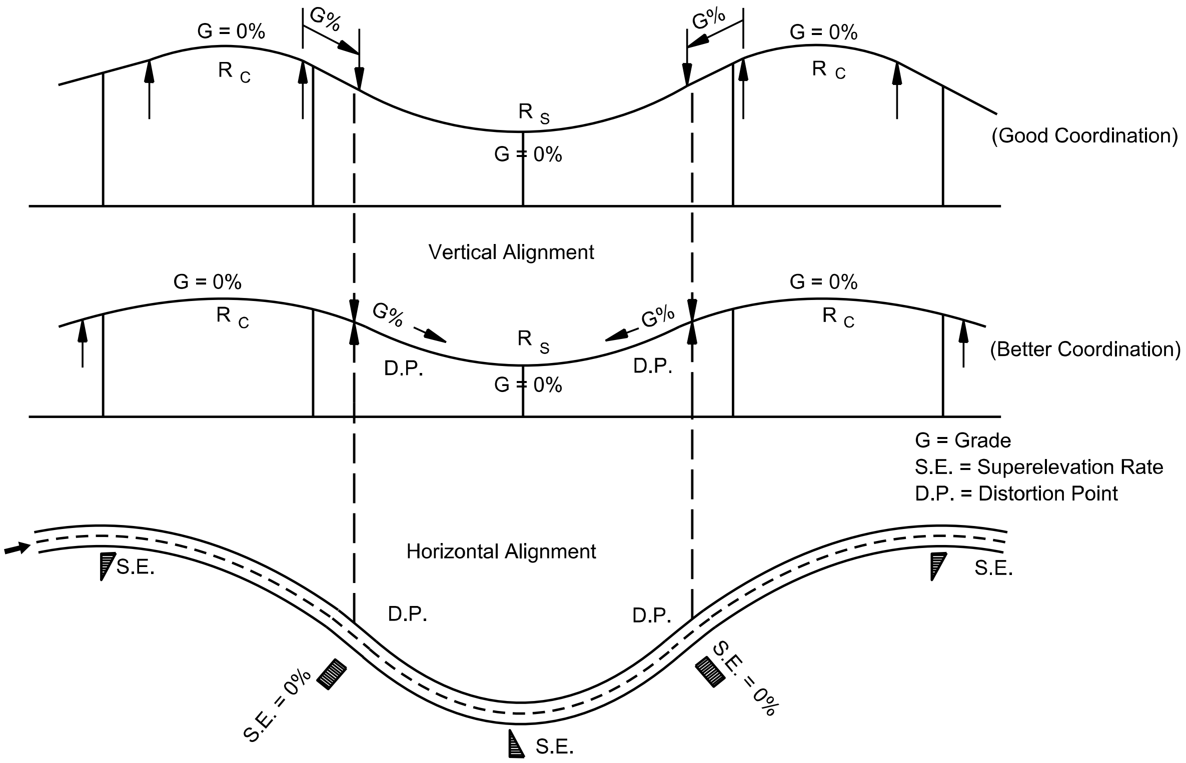
Figure 6-17: Superimposition of Horizontal and Vertical Alignments



a) Horizontal Alignments

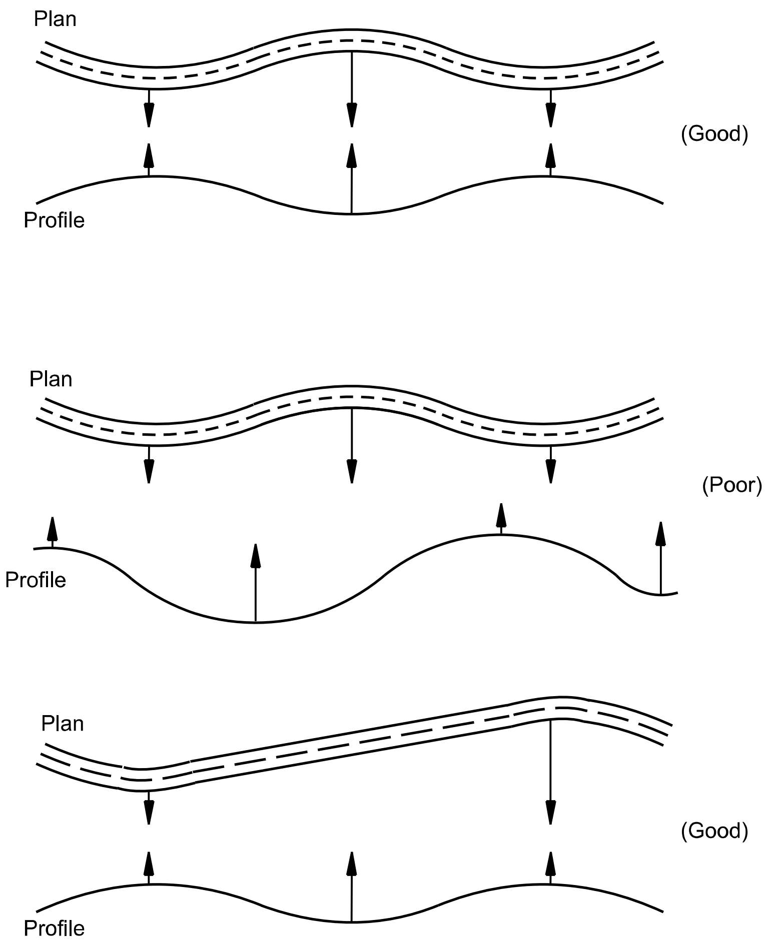


b) Vertical Alignments: (Relation $R_C : R_S$)

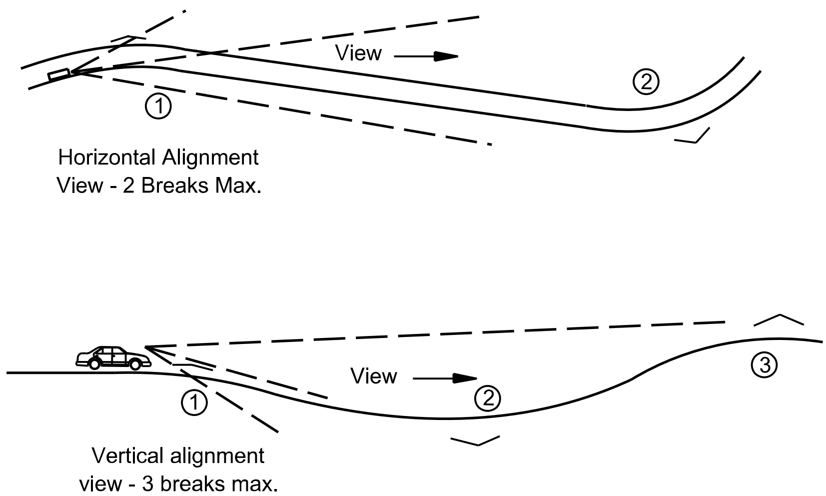


c) Coordination of Distortion Points in Horizontal and Vertical Alignments

Figure 6-18: Examples of Superimposition of Horizontal and Vertical Alignments



a) Coordination Of Horizontal And Vertical Alignment



b) Maximum Viewable Alignment Breaks

6.6.4 Profile Gradelines

The profile gradeline of a roadway typically has the greatest impact on a facility's cost, aesthetics, safety, and operation. The profile is a series of tangent lines connected by parabolic vertical curves. It is typically placed along the roadway centreline of undivided single carriageways and at the two median edges of the travelled way on dual carriageways.

The designer must carefully evaluate many factors when establishing the profile gradeline of a roadway. These include:

- maximum and minimum gradients;
- sight distance criteria;
- location of bridges and drainage structures;
- drainage considerations;
- water table elevations;
- location through intersections and interchanges;
- roadway/railroad crossings;
- types of soil;
- blowing sand;
- adjacent land use and values;
- roadway safety;
- coordination with other geometric features (e.g. the cross section);
- topography/terrain;
- truck performance;
- available right-of-way;
- type and location of utilities;
- urban/rural location;
- aesthetics/landscaping;
- construction costs;
- environmental impacts;
- driver expectations;
- intersection profile treatment; and
- pedestrians, disabled accessibility, and cyclists in urban areas.

6.6.4.1 Separate Profile grade lines

- Separate grade lines should be considered for all divided roadways. The use of separate grade lines provides the opportunity to optimize the vertical alignment, drainage features, and provide a safer more economical design.
- They are not normally considered appropriate where medians are less than 18m wide. Exceptions to this may be minor differences between opposing grade lines in special situations.
- In addition, for either interim or ultimate primary roadways, any appreciable grade differential between roads should be avoided in the vicinity of at-grade intersections. For traffic entering from the crossroad, confusion and wrong-way movements could result if the pavement of the far roadway is obscured because of excessive differential.

6.7 Emergency Escape Ramps

6.7.1 Purpose and Need

Long, steep downgrades can result in the drivers of heavy vehicles losing control and it is desirable to take measures to prevent the occurrence and limit the consequences of runaway heavy vehicles. Out-of-control vehicles typically result from drivers losing control because of the loss of brakes through overheating or mechanical failure, or because the driver failed to change down gears at the appropriate time. When considering the provision of runaway vehicle facilities, it is suggested the client liaise with stakeholders with respect to the location, spacing, and design of the facilities. Measures aimed at managing errant vehicles on steep descents include:

- alerting drivers of a steep descent on the approach to the downgrade,
- regulating use of a low enough gear to control the descent speed of heavy vehicles, and
- providing containment facilities for runaway vehicles.

Provide traffic signing to warn drivers of steep descents and to instruct drivers to use a low gear. See Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) for guidance on signing.

6.7.2 Containment Facilities

Runaway vehicle containment facilities include gravity safety ramps, arrester beds, or dragnet. A combination of these facilities may be required to suit a particular site. In addition, in some cases it may be desirable to place an energy absorbing barrier at the end of a safety ramp or arrester bed to cover an event where a vehicle has not totally decelerated within the ramp or bed (e.g. natural compaction of bed material reduces its effectiveness).

6.7.2.1 Gravity Safety Ramp

Gravity safety ramps use an ascending grade to reduce the speed of a runaway vehicle. Normally, ramps are hard surfaced and take advantage of naturally occurring grades in mountain ranges.

6.7.2.2 Arrester Beds

Arrester beds are long trenches filled with small round gravel particles (pea gravel) that are designed to stop runaway trucks. The truck is stopped by drag and friction as the vehicle sinks into the pea gravel bed.

6.7.2.3 Dragnets

A dragnet vehicle-arresting barrier consists of a chain link net that is attached to energy absorbing poles. Several nets in series are needed to capture heavy vehicles. Design of dragnet systems needs to be in accordance with the manufacturer’s design parameters.

6.7.3 Warrant for Investigation

Downgrades have the potential to cause brake failure in heavy vehicles and should be considered for treatment to reduce the risk of runaway vehicles. Grade and length combinations that warrant investigation are shown in Table 6-10.

Where a warrant has been established for investigation for treatment of a steep downgrade, use the design process in Section 6.7.5.

Table 6-10: Typical Warrants for Analysis for Runaway Vehicles

Grade	Minimum Continuous Length (km)
-3%	8.0
-5%	3.1
-7%	1.9
-9%	1.4
-12%	1.0

Source: (2)

6.7.4 Location and Spacing

Do not construct emergency escape ramps where an out-of-control vehicle will need to cross oncoming traffic. On undivided roads, safety ramps should ideally be located at the start of a left-hand curve as the runaway vehicle can readily negotiate a tangential path into the ramp. On divided roadways where adequate space is available in the median, safety ramps can be located on either side of the roadway if adequate advance warning signs are erected prior to the safety ramp exit.

Locate safety ramps prior to or at the start of the smaller radius curves along the alignment. For example, an escape ramp after the tightest curve will be of little benefit if trucks are unable to negotiate the curves leading up to it.

Vehicle brake temperature is a function of the length of the grade; therefore, escape ramps are generally located within the bottom half of the steeper section of the alignment.

Lack of suitable sites for the installation of ascending type ramps may necessitate the installation of horizontal or descending arrester beds. Suitable sites for horizontal or descending arrester beds can also be limited, particularly if the downward direction is on the outside or fill side of the roadway formation.

For new projects, Table 6-11 may be used as a guide when considering the need for escape exits on grades greater than 6% and with the number of trucks exceeding 150 per day.

Table 6-11: Approximate Distance from Summit to Safety Ramp

Grade (%)	Approximate Distance from Summit to the Ramp (km)
6 – 10	3.0
10 – 12	2.5
12 – 15	2.0
15 – 17	1.4
17	1.0

Note: Actual distance will depend on site topography, horizontal curvature, and costs.

Source: (2)

The distances in Table 6-11 are not absolute and greater distances may be desirable based on site location and other factors. The need for a facility will be increased if the number of trucks is more than 250 per day and the maximum decrease in operating speed between successive geometric elements is approaching the limits set in Table 6-12.

Table 6-12: Maximum Decrease in Speed Between Successive Geometric Elements

Grade (%)	Maximum Decrease in Speed Between Successive Geometric Elements (km/h)
< 6	10
6 – 10	8
> 10	6

Source: (2)

6.7.5 Design Process

6.7.5.1 Step 1 – Vehicle Entry Speed

The selected design vehicle should be used in determining the vehicle entry speed to the facility. Heavy runaway vehicles attain high speeds, but speeds in excess of 130 km/h to 140 km/h will rarely be attained. An escape ramp should therefore be designed for a minimum entering speed of 130 km/h with a 140 km/h design speed desirable. Several formulae and software programs have been developed to determine the runaway speed at any point on the grade or the designer may use Figure 6-7. These methods can be used to establish a design speed for specific grades and horizontal alignments.

6.7.5.2 Step 2 – Truck Stability on Approach

Check the alignment of all curves preceding the ramp to ensure that a runaway vehicle can safely negotiate and will not roll over on curves beyond of the runaway containment facility. The maximum cornering speed is given by Equation 6.37.

$$\mathbf{v} = \sqrt{(\mathbf{agR})}$$

Equation 6.37: Maximum Cornering Speed

where: v = speed of vehicle, m/s
a = maximum lateral acceleration, 0.3 g
g = gravitational constant, 9.81 m/sec^2
R = radius of curvature, m.

If trucks are likely to roll over before reaching the containment facility then relocate the containment facility if the terrain allows.

6.7.5.3 Step 3 – Entry Alignment

The alignment of the escape ramp should be at a tangent or very flat curvature to reduce the likelihood that the driver will experience vehicular control problems. A 5-degree angle of departure or less is required, and as much sight distance as possible should be provided. The leading edge of the arrester bed must be normal to the direction of entry to ensure that the two front wheels of the vehicle enter the bed simultaneously.

6.7.5.4 Step 4 – Type of Facility

The constraints imposed by the terrain will largely determine the type of facility to be implemented. Several iterations of design may be necessary if a combination of facility types proves to be necessary. Changes to the type of facility and pavement type may be necessary to determine the best fit to the site constraints.

6.7.5.5 Step 5 – Facility Surface

The rolling resistance of the ramp will have a significant influence on the length required for the containment facility. Use the values shown in Table 6-13 to determine length calculations.

Table 6-13: Rolling Resistance of Roadway Surfacing Materials

Surfacing Material	Rolling Resistance (kg/1 000 kg GVM)	Equivalent Grade (%)[a]
Portland cement concrete	10	1.0
Asphalt concrete	12	1.2
Gravel, compacted	15	1.5
Earth, sandy, loose	37	3.7
Crushed aggregate, loose	50	5.0
Gravel, loose	100	10.0
Sand	150	15.0
Pea gravel	250	25.0

Note: Pea gravel is rounded gravel having a uniform particle size of about 10 mm.

Source: (1)

Rounded particles (e.g. uncrushed pea gravel) with uniform gradation produce higher deceleration than the more angular crushed aggregate. This is because the vehicles sink deeper into the rounded gravel, transferring more energy to the stones over a shorter length. The use of a material with low shear strength is desirable in order to permit tire penetration.

Crushed stone may be used, but is not considered effective as it will require longer beds and will need regular fluffing or de-compaction.

Sand has problems of blowing, drainage, compaction, and contamination and should not be used unless alternative materials are unavailable. Beds using sand will require a strict maintenance regime to ensure their continued effectiveness. However, all arrester beds and bedding materials require regular maintenance.

Nominal 10-mm pea gravel has been used satisfactorily in testing. The gravel should be predominantly rounded, of uniform gradation, free from fine fractions and with a mean particle size ranging between 12 mm and 20 mm. In general, gravels with a smaller internal friction angle will perform better than those with larger internal friction angles.

6.7.5.6 Step 6 – Facility Length

The length of a containment facility will vary depending on entry speed, grade, pavement surface and the type of facility.

The vehicle entry speed described in Section 6.7.5.1 is used as the initial velocity for determining the length of an arrester bed. The length of an arrester bed is determined by Equation 6.38.

$$L = \frac{Vi^2 - Vf^2}{2.54(R + G)}$$

Equation 6.38: Arrester Bed Length

where: L = length travelled, m
Vi = initial velocity, km/h
Vf = final velocity, km/h
R = grade, %
G = rolling resistance expressed as a grade from Table 6-13, % (+ for upgrades and – for downgrades)

Where there is a grade change in the arrester bed, Equation 6.38 can be used to calculate the length required on each grade. The final velocity of a section becomes the initial velocity of the next section.

6.7.5.7 Step 7 – Facility Design

1. General. This step requires the preparation of the layout and design of the facility. Several iterations with different combinations of facility types may be necessary.

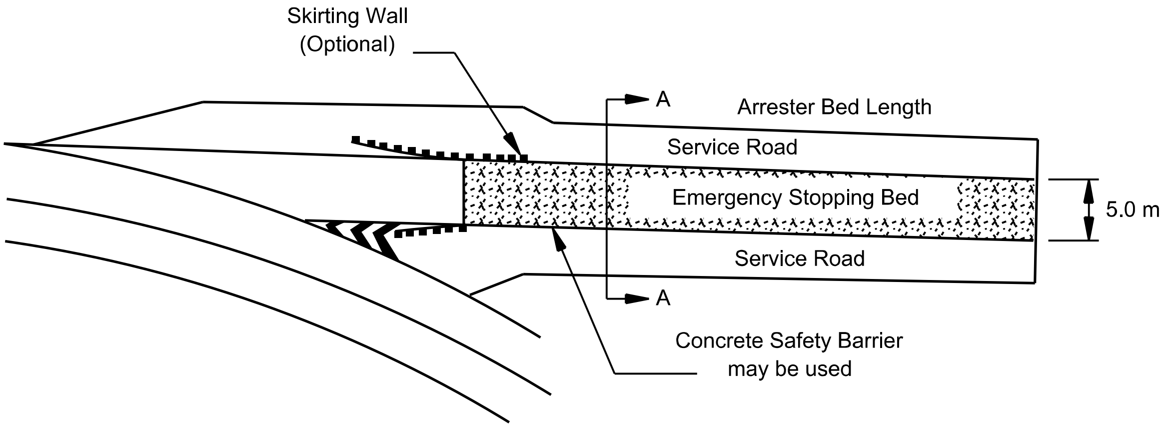
2. Safety Ramp Design. The grade of the safety ramp will be largely determined by the terrain. Safety ramps need steep-sided cut batters on both sides. When a runaway vehicle stops in a ramp, it will begin to roll back because the brakes are not functional. In this situation, drivers must jack-knife the vehicle against the sides of the ramp to prevent it rolling down the ramp.

3. Arrester Bed Design. Arrester beds aim to provide deceleration similar to an emergency braking situation to avoid the risk of the truck cabin being crushed by a shifting load. Arrester beds can be constructed on up, level, or downgrades depending on the topography at the site. Arrester beds on downgrades require additional length to bring out-of-control vehicles to rest. An example layout of an arrester bed is shown in Figure 6-19.

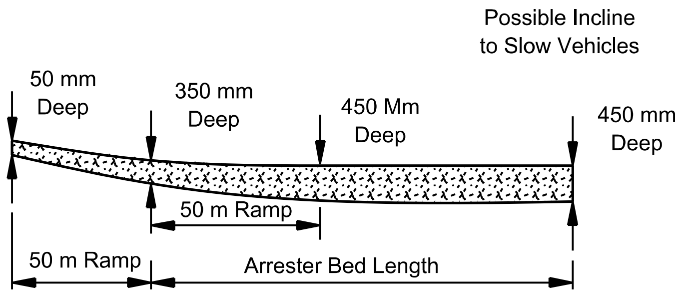
A working area for a retrieval vehicle or crane may be provided on one side of the bed or on both sides as shown in Figure 6-19. An access area on both sides of the arrester bed will not be necessary in many cases, but may be required where heavy vehicles on a route carry very heavy or difficult loads, which will require retrieval vehicles or cranes to work from both sides.

Table 6-14 provides a summary of the features associated with arrester beds and key considerations required in design.

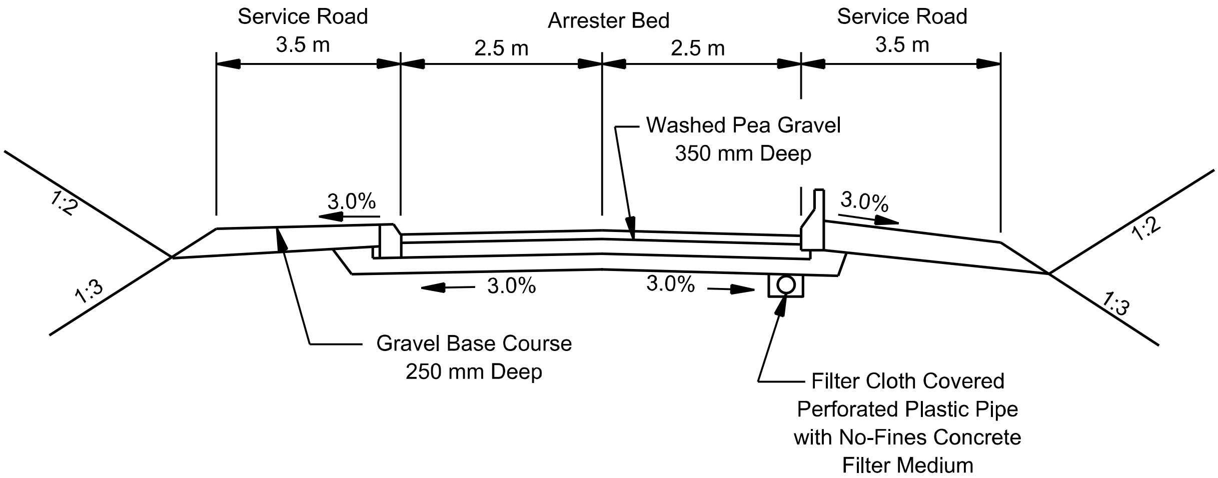
Figure 6-19: An Example of an Arrester Bed Layout



Plan



Section Along The Road



Source: (2)

Cross Section A-A

Table 6-14: Design Features of Arrester Beds

Feature	Consideration
Horizontal Alignment	Some steering is possible in a gravel arrester bed. Where a curve is necessary, the radius should be generous, well in excess of standard travel speed to radius ratio.
Lateral Location	The round gravel sprayed or dislodged from the arrester bed may be a hazard to passing vehicles as it is likely to cause crashes due to loss of steering and traction on adjoining traffic lanes. An arrester bed should be located at least 4 m or more from the travel lanes. This offset provides an access area for recovery vehicles and provides a space for containment of sprayed gravel. An alternative is to provide a curb at an offset to the travel lane to contain gravel that would be swept onto travel lanes. Cubs should not prevent a grader with a blade extension to regrade the gravel after a vehicle has been removed from the arrester bed.
Width	A width of 5 m gives some room for steering yet should control a heavy vehicle if it starts to get out of control within the bed.
Depth	A gradual or staged increase in the depth of the bed should be provided on the entry ramp. There should be a gradual increase in aggregate depth in the first 30 m although the initial depth of the aggregate need not start at zero. This gradual increase also assists in vehicle extraction. A maximum bed depth of 350 mm provides adequate deceleration without causing damage to the vehicle. Higher deceleration rates can be achieved by increasing the bed depth up to 450 mm; however, driver safety may be jeopardised and damage may be caused to the vehicle. An increase in depth to 450 mm depth at the end of the bed will provide for higher-speed vehicles to be arrested at the point where vehicle speed has been reduced by the treatment.
Base	The base of an arrester bed should be concrete with a cross slope of 2% towards the concrete barrier and a drainage system. Steeper cross slopes should not be used as they can cause trucks to veer off-line as they pass through the arrester bed.
Barrier	A vertical concrete barrier should be placed on the edge of the bed furthest from the travel lane to assist in keeping vehicles travelling along the bed.
Drainage	Storm water should be directed away from the bed. Design the base of the bed to accommodate drainage and avoid contamination of the arrester bed material by accumulation of fines that would compact the bed material. Installation of perforated drains in the base of the bed and lining the bed base and sides with asphalt or cemented material is required.
Fuel Spill Containment	Truck fuel lines may be ruptured when impacting the gravel in an arrester bed. The drainage system of the arrester bed should be fitted with a fuel spill containment facility.
Arrester Bed Material	Rounded pea gravel in loose condition is essential to make an arrester bed effective. The aggregate should be predominantly single sized and uniform. It should be clean, free of fines and have smooth rounded surfaces. Deceleration characteristics of the bedding material may be affected by wet weather.

Source: (2)

6.7.5.8 Step 8 – End Treatment

Where the ramp length is inadequate to fully stop an out-of-control vehicle, a positive attenuation or last chance device may be required. Ensure that the device does not cause more problems than it solves (e.g. sudden stopping of the truck can cause the load to shift with potentially harmful consequences to the driver and the vehicle). Judgement will be required on whether the consequences of failing to stop are worse than these effects. Crash cushions or piles of sand or gravel may be used as last chance devices.

A dragnet system may be needed if a “fail-safe” end treatment is not available.

6.7.5.9 Step 9 – Vehicle Recovery Facilities

Access and anchors for cranes and/or tow trucks should be designed and provided to facilitate removal of the disabled vehicle from the containment facility. The design of removal facilities should ensure the occupational health and safety of removal workers.

1. <u>Safety Ramp Recovery Facilities</u>. If separate access is available to the top of the ramp a tow truck can be used for vehicle recovery. A large anchor block should be buried below the surface at the top of the ramp, as recovery will be assisted if the tow truck can be chained to an anchor while winching the runaway vehicle up the steep slope.

If access is not available to the top of the ramp it will be necessary to use a bulldozer to engage the rear of the runaway vehicle and lower it backwards down the slope and hence the treatment should be designed for this loading.

2. <u>Arrester Bed Recovery Facilities</u>. To facilitate recovery of vehicles from an arrester bed, provide a service road adjacent to the bed with a minimum width of 3.5 m. Access to the service road should be available for either two heavy-duty tow trucks or two 50-tonne capacity cranes. Therefore, the pavement of the service road should be capable of supporting 50-tonne capacity cranes. The service road should also be designed so a grader with blade extension can grade the gravel after a vehicle has been removed.

Anchor blocks are required to secure tow trucks while winching vehicles out of the arrester bed. Locate anchor blocks at 35-m intervals along the service road and 10 m from the entry and end of the arrester bed. Anchors should be designed to a 35-tonne winching force through an attachment shackle rated to withstand the design load. Attachment shackles should be recessed flush with pavement levels. It is preferable that recovery is made from the exit end of the arrester bed, as articulated vehicles will jack-knife if dragged backwards through the bed.

To enable drivers to get assistance, CB radio frequency or telephone numbers for emergency service may be advised on signposting adjacent to the bed or service road.

6.7.5.10 Step 10 – Design Delineation

The existence and location of a containment facility must be made obvious by signage to give the operator of an out-of-control vehicle time to react and decide to enter the facility. Standard signs should be provided and located in accordance with Abu Dhabi *Manual on Uniform Traffic Control Devices* (22). The location of signs, street lighting poles, and overhead power lines should not obstruct the operation of the arrester bed or retrieval operations. Routine maintenance of any light poles should not impose any entry restriction to the arrester bed at any time. Adequate delineation should also be provided so that the entrance to a containment facility is not mistaken for the through roadway and the entry path to the facility is clear by day and night.

6.7.6 Brake Check Areas

Provide turnouts or pull-off areas at the summit of a grade to allow drivers to inspect equipment on the vehicle and to check that the brakes to ensure they are not overheated.

6.7.7 Maintenance

After each use, aggregate arrester beds should be reshaped using power equipment to the extent practical and the aggregate scarified as appropriate. Because aggregate tends to compact over time, the bedding material should be cleaned of contaminants and scarified periodically to retain the retarding characteristics of the bedding material and maintain free drainage.

7 RURAL AND URBAN FREEWAYS

7.1 Overview

Freeways are dual-carriageway arterials with full control of access (regulation of public access rights to and from properties adjacent to the freeway). Freeways usually provide for high levels of safety and efficiency in the movement of large traffic volumes at high speeds. With full control of access, priority is given to through traffic by providing access connections with selected adjacent public roads only and by prohibiting crossings at grade and direct private driveway connections.

The principal advantages of access control include safety, preservation of capacity, higher speeds, and lower than normal crash frequencies. Roadways with fully controlled access have grade separations at all railroads and either grade separations or interchanges at selected public crossroads. Other crossroads are either interconnected or terminated.

Essential freeway elements include:

- roadways;
- medians;
- grade separations at crossroads;
- ramps to and from the travelled way at selection locations;
- weigh-in-motion locations;
- petrol stations;
- rest areas; and
- in some cases, frontage/service roads.

Chapter 2 “General Design Criteria and Control,” Chapter 3 “Cross Section Elements, Chapter 4 “Sight Distance,” Chapter 5 “Horizontal Alignment,” and Chapter 6 “Vertical Alignment” describe geometric design elements, controls, and criteria that are also applicable to freeways. Chapter 12 “Interchanges” provides the design criteria for freeway interchanges. Further guidance on freeways can be found in the AASHTO *A Policy on Geometric Design of Highways and Streets* (1) and the Institute of Transportation Engineers *Freeway and Interchange Geometric Design Handbook* (36).

7.2 Freeway Systems

A freeway is an integral part of the overall roadway system. In rural areas, freeways provide a link between cities in generally a linear fashion (i.e. they connect from point A to point B without a major deviation in alignment). In urban areas, development, access, mobility, and environmental effects are major factors in the design of freeways. Freeways may pass through a city, terminate within the city, or by pass the city itself. These various freeway types along with arterials, collectors, and local streets form the integrate part of urban roadway network.

7.2.1 Basic Components

The following are the basic components that make up a freeway system:

1. <u>Freeway Segment</u>. The freeway segment is the freeway length between system interchanges. It is also part of the linear or freeway corridor system.

2. System Interchange. This is a freeway-to-freeway interchange. It allows a trip to be maintained on the freeway network without interruptions.

3. Service Interchanges. This is an interchange between the freeway and an arterial or collector, which provides local or area wide service.

4. Freeway Spur or Distributor. A freeway spur is an auxiliary controlled-access roadway serving an adjunct to a corridor freeway. It facilitates the collection and distribution of traffic a specific area.

7.2.2 Freeway Spacing

The following factors affect the network configuration and spacing of freeways:

1. Travel Requirements. Travel requirements have a major effect upon the location and spacing of freeways. The biggest determinant of freeway spacing is trip density. Vehicle trip origins per square kilometre are largely the by-product of types and intensities of land use, population densities, and car ownership. The nature of the interrelationship between these factors allows population density, as a substitute for trip density, to be used as a general planning guide in determining freeway spacing.

2. Physical Characteristics. Physical characteristics including terrain, watercourses, street patterns, through-route extensions, size and shape of metropolitan area, and historical development have a significant effect on the location of freeways and the shape of freeway networks. The physical characteristics together with the design features determine the more-detailed aspects of freeway location and spacing, rather than overall or average spacing of freeways.

3. Economical/Sociological/Environmental/Political Aspects. Economic, sociological, environ-mental, and political aspects, also determine when and where a freeway facility is provided. The requirements for freeways based on trip or population densities and guided by physical characteristics in the area, may not always be satisfied. If so, the freeway network with missing links becomes imbalanced, resulting in distorted travel patterns. This weakness places a greater traffic burden on those freeways system components that can be achieved, necessitating design that is more expensive and/or upgrading for freeway corridors.

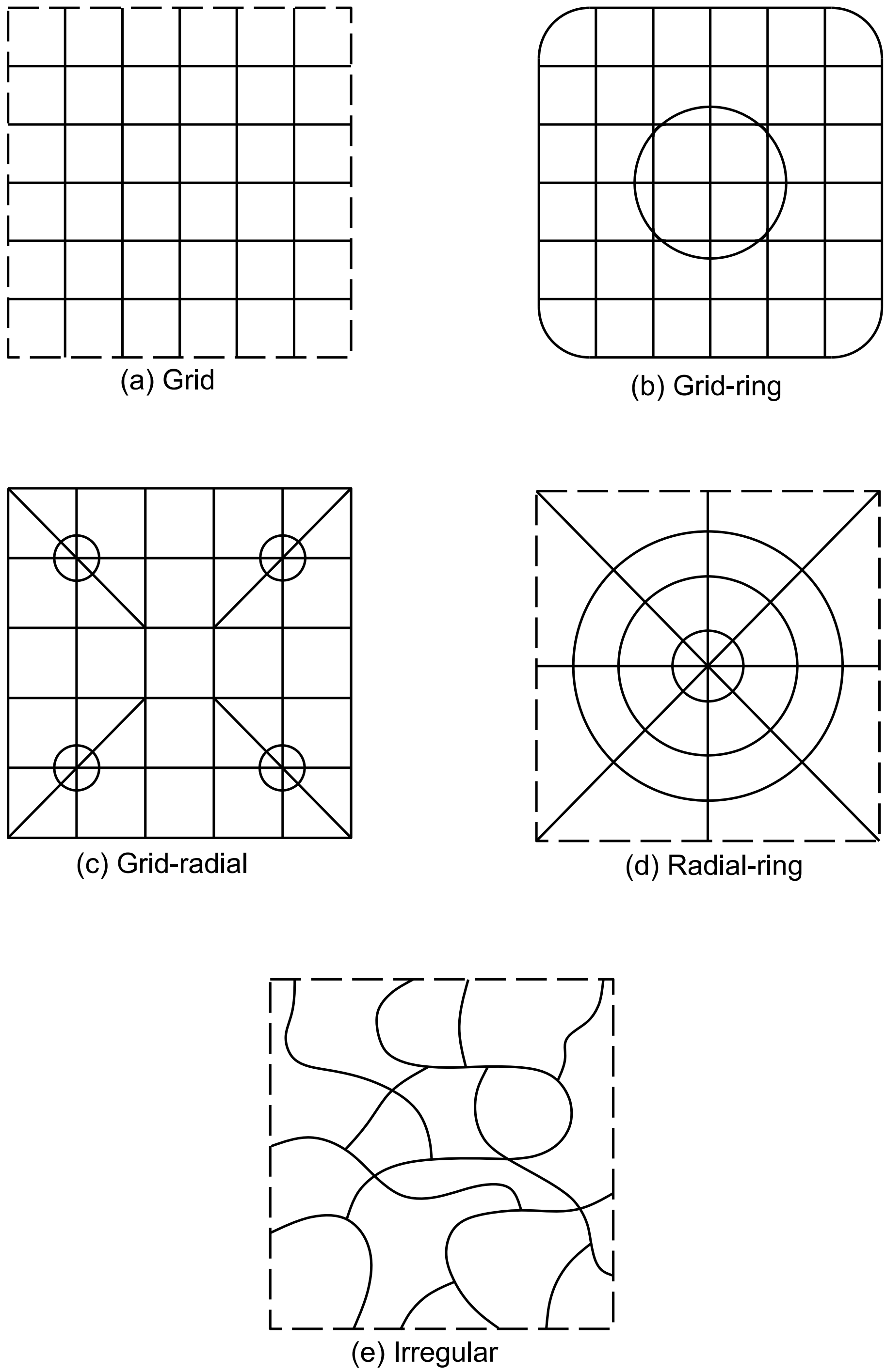
Studies have indicated that traffic movements in metropolitan areas of medium-to-large size generally require an ultimate network of freeways at optimal intervals of 5 km to 7 km in the central area and 7 km to 10 km in the outer areas. These indicated spacing values tend to distribute and balance traffic throughout the network, and with the proper sizing of freeways, could be expected to function into the future at acceptable levels of service.

7.2.3 Network Configurations

Freeway networks may be one of following basic forms shown in Figure 7-1:

1. Grid. The most versatile and efficient configuration is the grid network shown in Figure 7-1(a). It generally provides the most uniform distribution of traffic, continuity of routes, and well-balanced service to land areas.

Figure 7-1: Basic Freeway Networks



Source: (36)

2. Grid-Ring. The grid-ring system shown in Figure 7-1(b) is also highly efficient and has similar attributes to the grid.

3. Radial. Radial freeways shown in Figure 7-1(c) and Figure 7-1(d) pose problems with a high concentration of traffic near the city centre. Turning movements at interchanges tend to be greater and more difficult to handle than on the grid networks.

4. Irregular. The irregular network shown in Figure 7-1(e) is typically a by-product of rugged terrain. Without system regularity, it lacks the attributes of a grid system, although it can operate with some degree of efficiency if warped to simulate a grid system.

7.2.4 Linear Freeway Systems Configurations

Figure 7-2 illustrate the six basic linear freeway configurations that are applicable under appropriate conditions. These are further described as follows:

1. Conventional. A conventional configuration, Figure 7-2(a), is a freeway without supplementary parallel roadways (i.e. a facility that relies on the interchange of traffic directly with intersecting or traverse roads).

2. Two-Way Frontage Roads. This configuration is generally only applicable in rural areas. In most cases, the two-way frontage roads intersect the crossroad external to the interchange. It may also be integrated with the interchange ramps as shown in Figure 7-2(b).

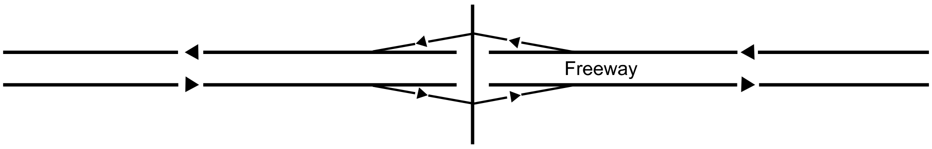
3. One-Way Frontage Roads. This freeway system arrangement has continuous or near continuous one-way frontage roads, Figure 7-2(c). This configuration provides collection and distribution of traffic with the surrounding areas and provides flexibility in the use of successive interchanges.

4. Two-Way/One-Way Frontage Roads. This configuration is a combination of a one-way frontage road concept through interchanges with two-way frontage roads between the interchanges. See Figure 7-2(d).

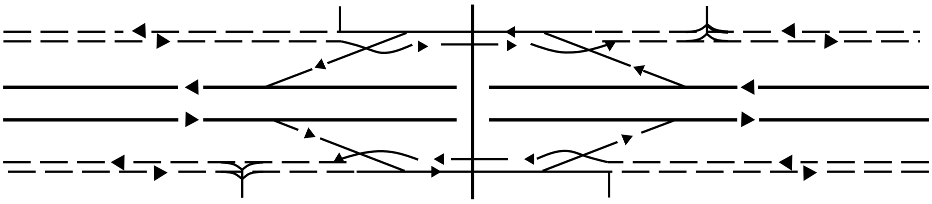
5. Combination Frontage Roads and Collector-Distributor (C-D) Roadways. This system uses continuous one-way frontage roads that are transformed into C-D roadways in the vicinity of interchanges. See Figure 7-2(e).

6. Continuous C-D Roadways. In this configuration, fully controlled access roadways (C-D roadways) closely paralleling the basic freeway produces, in effect, two freeway facilities within the same transportation corridor. See Figure 7-2(f).

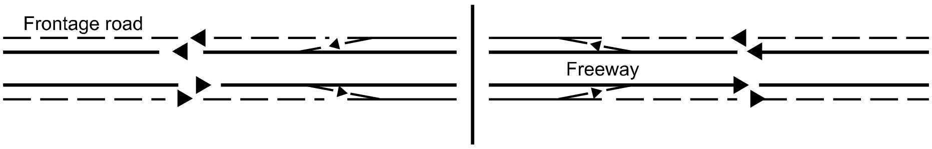
Figure 7-2: Linear Freeway System Configurations



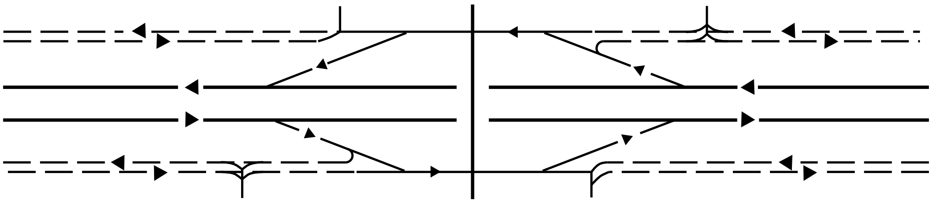
(a) Conventional



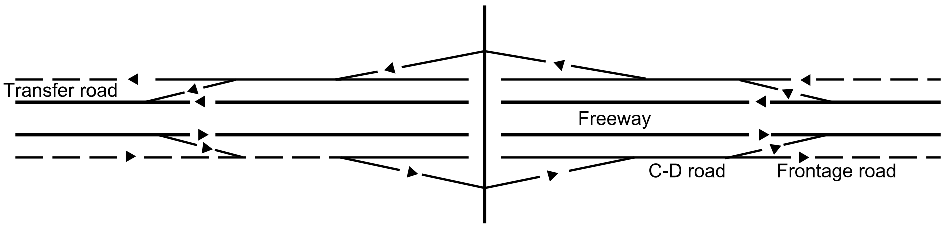
(b) Two-Way Frontage Roads



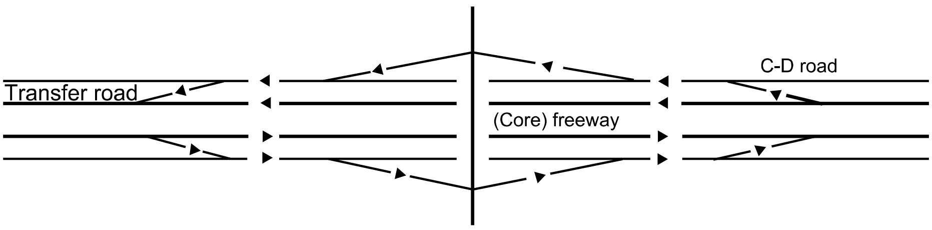
(c) One-Way Frontage Roads



(d) Two-Way/One-Way Frontage Roads



(e) Combination Frontage Roads and C-D Roads



(f) Continuous C-D Roads

Source: (36)

7.3 Freeway Types

7.3.1 Ground-Level Freeways

7.3.1.1 General Characteristics

In most areas of Abu Dhabi, the terrain is relatively flat. As a result, most freeways, and in particular rural freeways, are constructed essentially at ground level. See Figure 7-3. A major consideration in the design of ground-level freeways is the change in profile of each crossroad as it passes over or under the freeway; see Figure 7-4. Consequently, substantial lengths of ground-level freeways are generally not practical in heavily developed urban areas because the profiles of crossroads cannot be altered without severely impacting the community.

Ground-level freeways usually are employed in outlying sections of urban areas and rural areas where right-of-way is not as expensive. As a result, the cross sectional elements (e.g. medians, outer separations, borders) are widened to enhance the roadside design and appearance of the freeway.

In developed urban areas, it is usually desirable to provide continuous one-way frontage (service) roads on each side of the freeway that serve as a means of access to and from streets that are not carried across the freeway. In some situations, two-way service roads may be the only practical to maintain local service, even though they are less desirable than one-way service roads.

7.3.1.2 Typical Cross Section

Figure 7-5(a) illustrates a typical cross section for a ground-level freeway with service roads, and Figure 7-5(b) shows a typical cross section without service roads. Where additional right-of-way is available, the outer separations and borders should be widened to provide utility service strips and to insulate the freeway from the surrounding area. In areas where ramp connections are made to service roads, the width of outer separations should be increased to allow space for the design of ramps and ramp terminals.

Figure 7-6(a) illustrates restricted cross sections for ground-level freeways with a two-way service road. Figure 7-6(b) presents a restricted cross section without service roads.

7.3.2 Depressed Freeways

7.3.2.1 General Characteristics

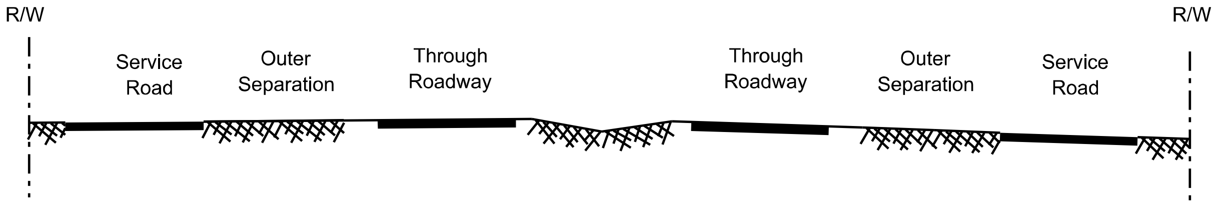
In urban areas, it may be desirable to depress the freeway below the ground level. Depressed freeways are less conspicuous than ground-level or elevated freeways, permit surface streets to cross at their normal grade (see Figure 7-7), and reduce freeway noise. However, these advantages have to be balanced against the increased cost of earthwork, drainage, and utilities. Pumping stations may be required to accommodate drainage.

An urban depressed freeway may occupy a full-block width and be parallel to the grid street system for most of its length. The roadways of a depressed freeway are typically located below the surface of the adjacent streets at an approximate depth of 6.50 m (to the closest feature on the ceiling) plus the depth for overhead structures (beam depth) and an allowance for future pavement overlays. For urban areas, the depth is to be taken as 6.00 m instead of 6.50 m as mentioned above.

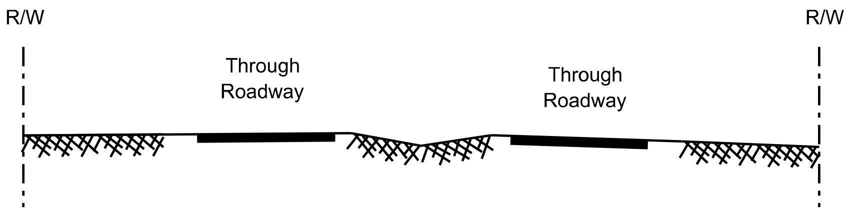
Figure 7-3: Typical Freeway at Ground Level

Figure 7-4: Typical Ground Level Freeway

Figure 7-5: Typical Cross Sections for Ground-Level Freeways

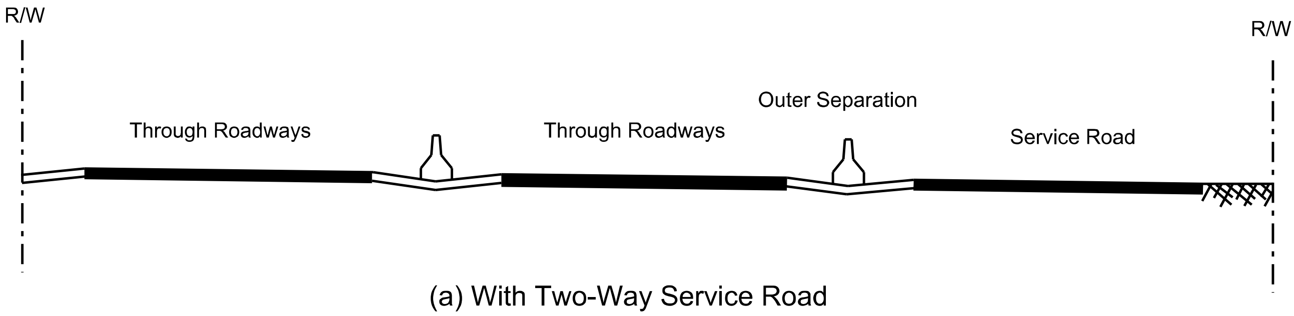


(a) With One-Way Service Roads

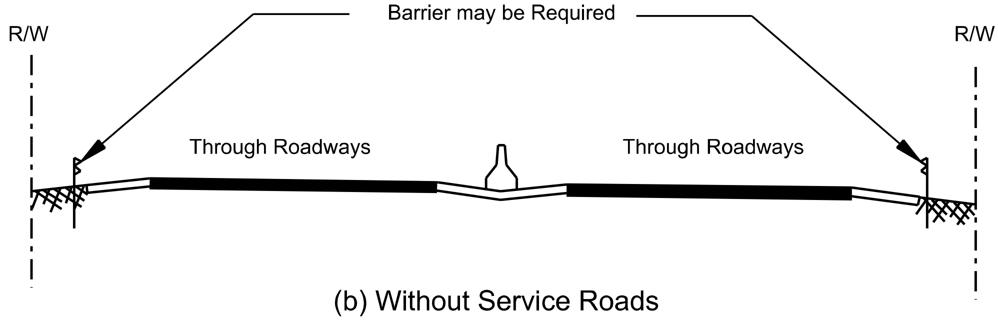


(b) Without Service Roads

Figure 7-6: Restricted Cross Sections for Ground-Level Freeways



(a) With Two-Way Service Road



(b) Without Service Roads

Note: For cross section widths, see the geometric design tables in Sections 7.6 and 7.7.

Source: (1)

Figure 7-7: Depressed Freeway (Sheikh Zayed Street Tunnel)

Depressed freeways are often flanked on one or both sides by service roads at the street level. All major streets pass over the freeway while other minor streets are intercepted by service roads or terminated at the right-of-way line. Access to surface streets is accomplished by ramps that connect directly with service roads or the crossing street where no service road exists. For interchange design guidance, see Chapter 12 “Interchanges.”

Provide fencing on structures passing over the depressed freeway and for retaining walls located in close proximity to the travelled way to reduce the possibility that objects will be dropped or thrown onto vehicles below.

7.3.2.2 Slopes and Walls

Often side slopes for depressed freeways are controlled by the available right of way. The designer should meet the slope criteria presented in Section 7.7 for urban freeways. Front slopes, if used beyond the shoulder, should be traversable. However, front slopes are typically not used beyond the shoulder on depressed freeways. In these cases, back slopes should not be steeper than 1V:3H.

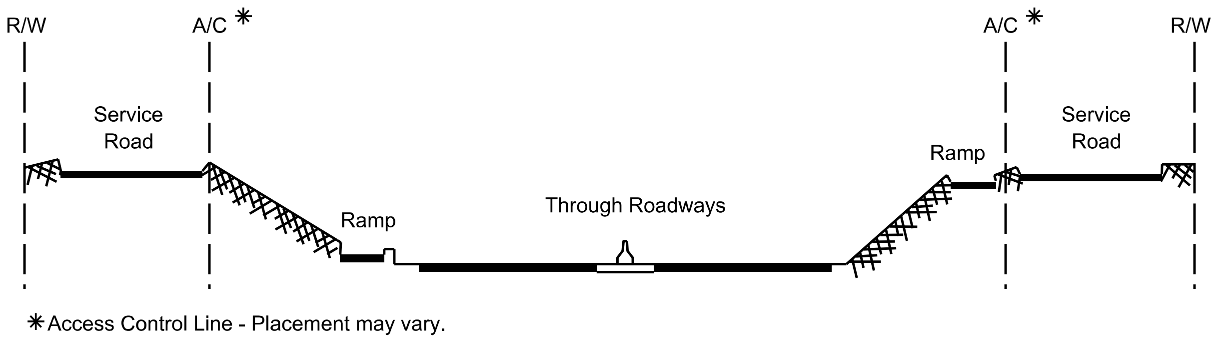
In developed areas, space may not be available for desirable slopes, particularly where ramps are present, and full- or partial-height retaining walls may be required. Do not locate retaining walls any closer to the roadway than the outer edge of shoulder and preferably be 0.6 m beyond the outer edge of shoulder. Where the wall is located at or near the shoulder edge, bridge columns, light fixtures, and sign supports should not protrude from the face of the wall.

7.3.2.3 Typical Cross Section

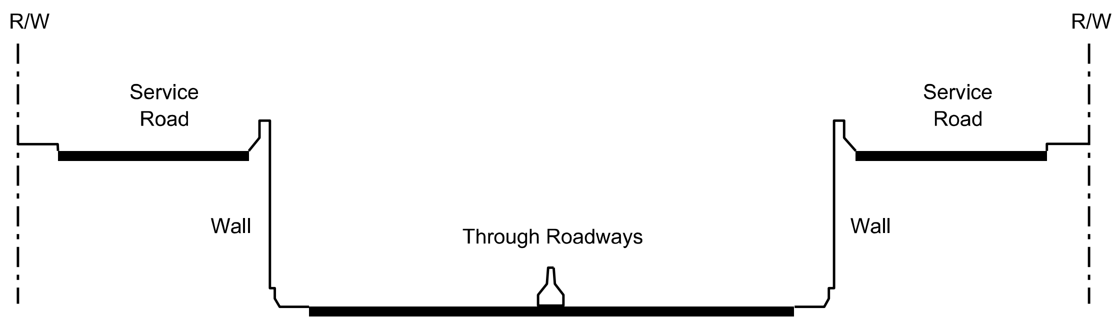
Cross sections of depressed freeways may vary through urban areas, in particular, the number of traffic lanes required. Another important factor is availability of right-of-way, which depends on the type and value of urban development, topography, soil and drainage conditions, and the frequency and type of interchanges to be used. The cross section should meet the design criteria presented in Sections 3 and 7.7 for urban and rural freeways, respectively. Where there are physical or economic constraints, it may be appropriate to vary certain design elements to fit the cross section within the available right-of-way. Figure 7-8(a) shows a depressed freeway with adequate right-of-way.

Figure 7-8(b) illustrates a depressed freeway with retaining walls in restricted right-of-way. Although the restricted cross section shown is acceptable, it should be used only where additional right-of-way would be extremely costly or where this type of cross section is necessary to preserve the surrounding environment.

Figure 7-8: Cross Sections for Depressed Freeways



(a) Typical Cross Section for Depressed Freeways



(b) Cross Section with Restricted Right-of-Way

Note: For cross section widths, see the geometric design tables in Sections 7.6 *and* 7.7*.*

Source: (1)

7.3.3 Elevated Freeways

7.3.3.1 General Characteristics

In urban areas, elevated freeways should only be considered where restricted right-of-way, high-water table, extensive underground utilities, close pattern of streets to be retained, terrain, or other circumstances make construction of a depressed freeway undesirable and/or uneconomical. An elevated freeway may be constructed on either a viaduct or an embankment. Consider the following:

1. Viaduct. Viaduct designs are influenced by traffic demands, right-of-way, topography, foundation conditions, urban development, interchange needs, availability of materials, and economic considerations. Figure 7-9 shows a freeway on viaduct. Because of these multiple considerations, viaducts are the most difficult of all freeway types to fit harmoniously into the environment.

Support columns for viaducts are positioned to provide clearance on each side and to leave the ground-level area free for other use. This design has the following advantages:

- practically all cross streets can be left open with little or no added expense;
- existing utilities that cross the freeway right-of-way are minimally disturbed; and
- surface traffic on cross streets usually can be maintained during construction with few, if any, detours.

Figure 7-9: Freeway on a Viaduct (Al Wahda Street – Sharjah)

The space under the structure can be used for surface-street traffic, for parking, or for a tram or transit line. The area under the viaduct may also have a high potential value to the community for joint development or other use, which may range from playgrounds to major buildings. The disadvantages with the viaduct design include high costs for maintaining the structure and its closed drainage system, difficulty in obtaining a pleasant appearance, and need for added police protection in the undeveloped space beneath the structure.

2. Embankment. Freeways on embankments are feasible in outer urban areas where crossing streets are widely spaced and where wide right-of-way and fill material are available. An elevated freeway on an earth embankment should be of sufficient height to permit intersecting surface roads to pass under it. Usually, an embankment section occurs on a combination-type freeway in rolling terrain where excavation material from depressed portions is used for the embankment. A roadside barrier generally will be warranted when the embankment slopes are greater than 1V:3H. The sloped areas may be planted to improve the appearance of the freeway.

Where appropriate, the fill may be confined by partial- or full-height retaining walls on one or both sides. With retaining walls, the total width may be reduced to widths comparable to elevated structures on viaducts. Special wall treatment or planting of trees and shrubs may make retaining walls aesthetically pleasing.

7.3.3.2 Typical Cross Section

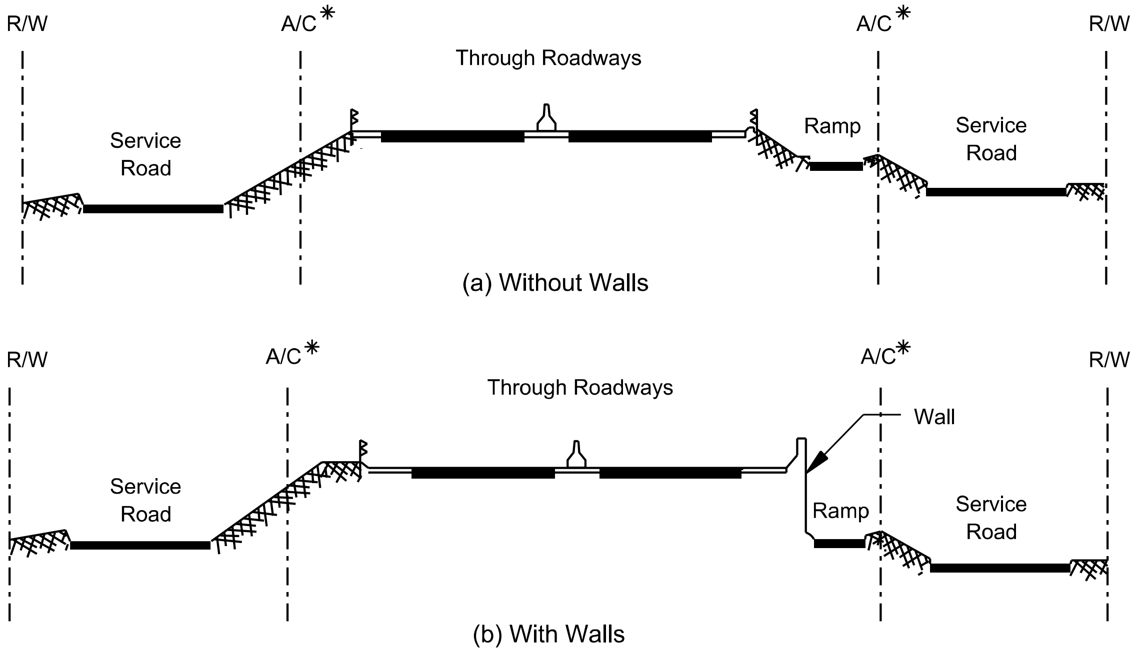
The total widths of elevated freeway sections can vary considerably. For elevated freeways on embankments, the total width is about the same as the total width needed for depressed freeways. Elevated freeways on structures may be cantilevered over parallel roadways or sidewalks.

Figure 7-10 illustrates an elevated viaduct freeway, where:

- Figure 7-10(a) shows the elevated freeway supported by columns on each side with at-grade service roads on the outsides of the columns.
- Figure 7-10(b) displays a cross section for a stacked freeway with the opposing traffic on top of each other. Due to the high cost and inability to expand the freeway in the future, this design should only be considered in very constrained right-of-way.
- Figure 7-10(c) illustrates where it may desirable to include separated special use lanes (e.g., HOV lanes, transit lanes) within a restricted right-of-way. The special use lanes are cantilevered over the through lanes for the freeway.
- Figure 7-10(d) shows typical cross section for where the through lanes are elevated on separate structures. It also shows ramps transiting in height between the elevated freeway and at-grade service roads.

Figure 7-11(a) shows a typical cross section for an elevated freeway on an embankment with at-grade service roads. Figure 7-11(b) shows the use a retaining wall for restricted conditions.

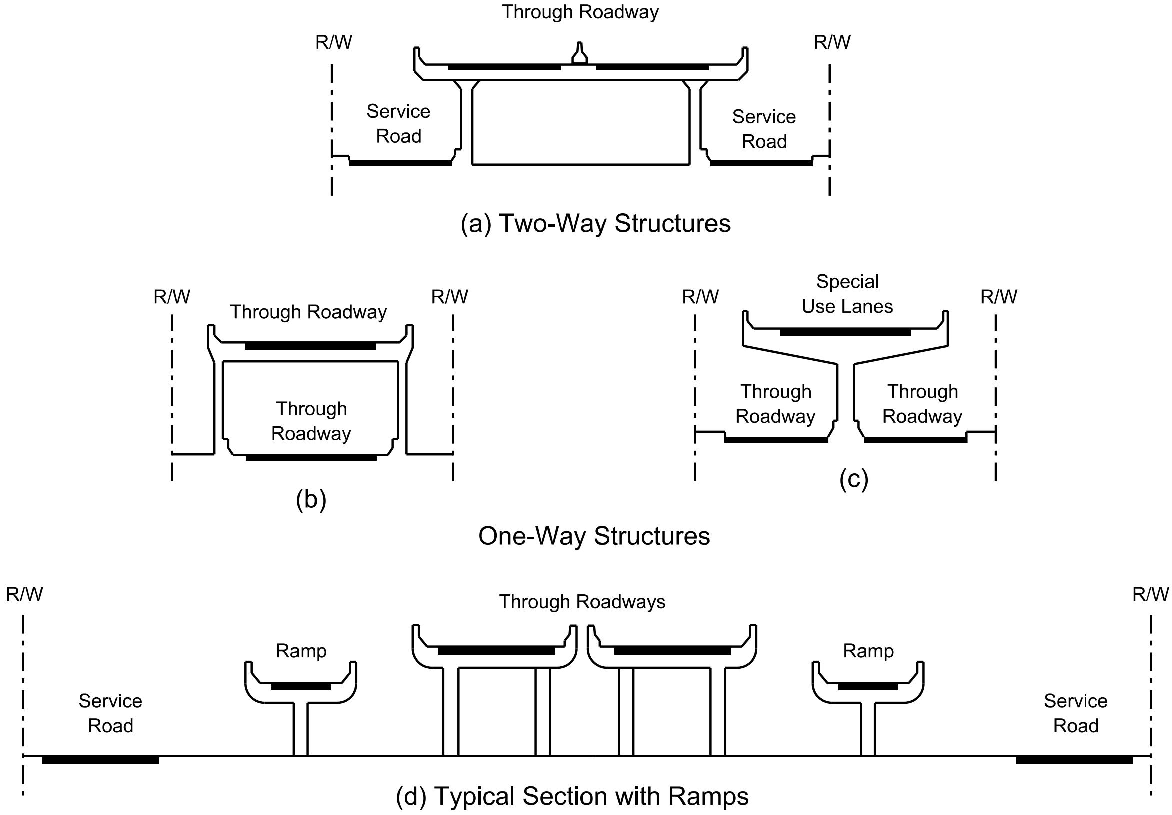
Figure 7-10: Typical Viaduct Cross Sections for Elevated Freeways



Source: (1)

* Access Control Line - Placement may vary.

Figure 7-11: Typical Embankment Cross Sections for Elevated Freeways



Note: For cross section widths, see the geometric design tables in Sections 7.6 and 7.7.

Source: (1)

7.3.4 Combination-Type Freeways

7.3.4.1 General Characteristics

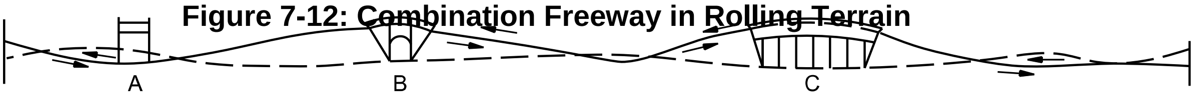
Typical, urban freeways will incorporate some combination of depressed, elevated, or ground level designs. Combination-type freeways result from variations in profile or cross section.

7.3.4.2 Profile Control

The terrain will generally determine the type profile control required. The designer should consider the following:

1. Rolling Terrain. The typical profile of a combination-type freeway in rolling terrain is shown in Figure 7-12. The best profile is usually developed by underpassing some cross streets and overpassing others. The facility is neither generally depressed nor elevated, although for short lengths it incorporates the design principles for fully depressed or fully elevated roadways. For example, in Figure 7-12, at A and C the facility is depressed, at B it is elevated on an earth embankment, and at each end at ground level.

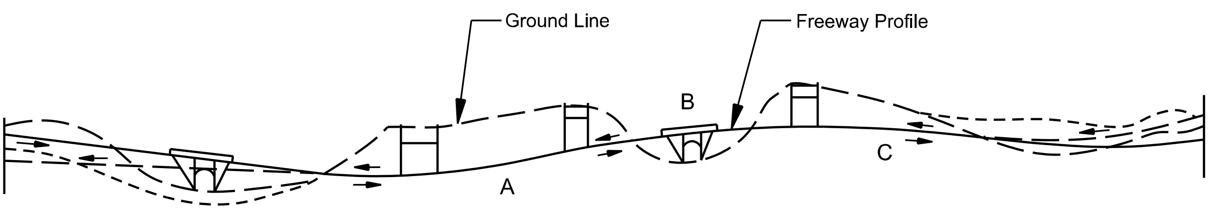
Figure 7-12: Combination Freeway in Rolling Terrain



Source: (1)

2. Level Terrain. Figure 7-13 shows a variation of a combination-type freeway in level terrain. The roadway profile closely follows the existing ground between the grade separation structures. The freeway also overpasses important cross streets by rolling the gradeline to the appropriate height above the surface streets; where practical, cross streets are carried over the freeway (as at A in Figure 7-13). This combination-type freeway design is suitable for level terrain where soil and groundwater conditions or underground utilities preclude depressing the freeway mostly below the existing ground, or continuous viaduct construction is too costly or is otherwise objectionable. The freeway may be carried over a cross street on an earth embankment with a conventional grade separation structure (as at B in Figure 7-13) or on a relatively long structure (as at C in Figure 7-13). The factors that control the profile design are the availability of fill material and the soil conditions. This freeway design permits parallel or diagonal ramps to be provided between the grade separations

Figure 7-13: Profile Control — Combination-Type Freeway in Level Terrain



Source: (1)

One of the disadvantages of this design is that a roller coaster-type of profile results where several successive cross streets are overpassed at close intervals. This situation is more pronounced on horizontal tangents where drivers can see two or more grade separations ahead. A moving vehicle that is ahead of a driver may disappear into dips and reappear again as the grade rises to a crest. Therefore, designer should design the profile to eliminate dips that would limit the sight distance. Ensure adequate sight distance is provided to the exit ramps. Where truck traffic is heavy, maximum grades of approximately 2% are desirable to prevent queuing at the base of the grade.

To minimise the overall rise and fall and make the rolling profile less pronounced, the cross streets may be depressed several meters below the ground surface and the freeway grade may be raised several meters above the ground level between grade separation structures. The profile may be further improved by raising some cross streets to overpass the freeway. Minimum governing distances for grade separation design are discussed in Section 7.5.2.3.

7.3.4.3 Cross-Section Control

The examples in Figure 7-14 are also considered combination-type freeways, but the primary influence on their design is the cross section. These special designs usually apply to relatively short lengths of roadway to meet specific conditions.

7.3.5 Special Freeway Designs

7.3.5.1 Reverse-Flow Freeways

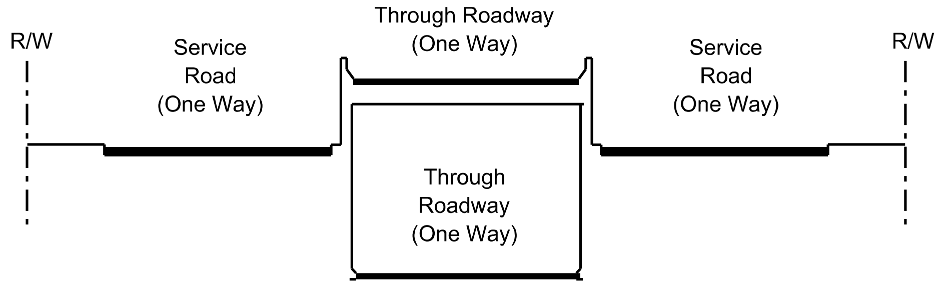
Special conditions may warrant the use of a reverse-flow roadway as a part of the freeway design. This is usually accomplished by placing a separate reversible roadway within the normal median area, as shown in Figure 7-15. Reverse-flow roadways are advantageous in that they may have unused capacity much of the time because of the limited numbers of access points. The costs of construction, maintenance, and operation of a freeway with a reverse-flow roadway may also differ considerably from those of a conventional freeway.

A separate reverse-flow roadway may be considered where:

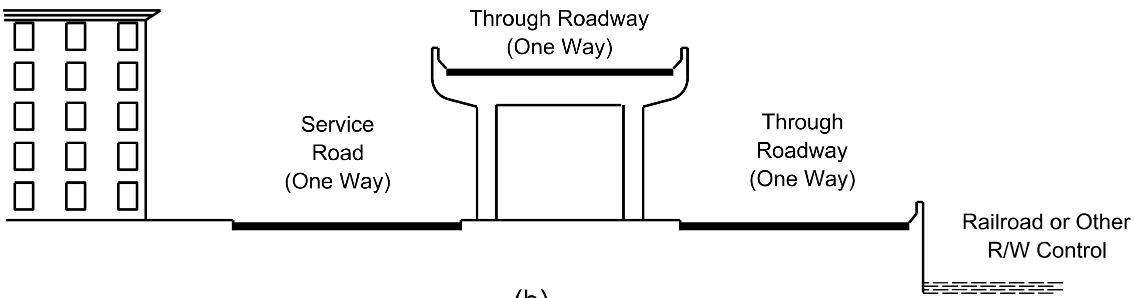
- the directional distribution during peak hours is substantially unbalanced (e.g. a 65:35 split) and capacity analysis indicates a need for a conventional facility more than eight lanes (4+4) wide;
- design controls and right-of-way limitations are such that providing two or more parallel facilities on separate rights-of-way is not practical; and
- a sizable portion of traffic in the predominant direction during peak hours is destined for an area between the central portions of the city or another area of concentrated development and the outlying area (i.e. a large percentage of peak-hour traffic travels a long distance between principal points of origin and destination with little or no need for intermediate interchanges).

In some areas, demand may be sufficiently great to justify the use of a reversible roadway exclusively for buses or other high-occupancy vehicles.

Figure 7-14: Combination-Type Freeways with Restricted Right-of-Way



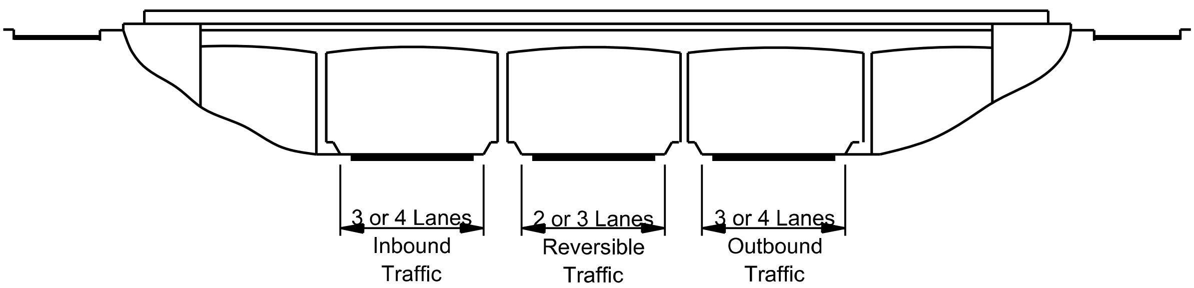
(a)



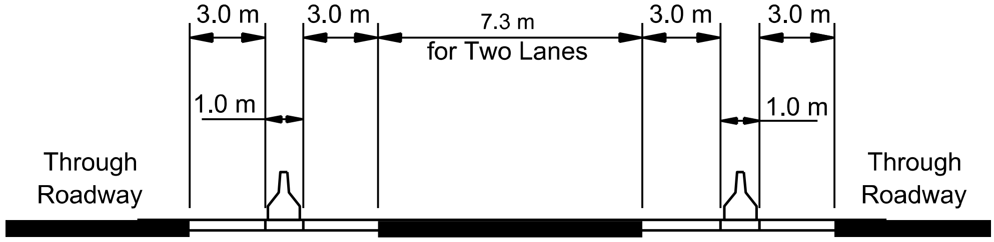
(b)

Note: For cross section widths, see the geometric design tables in Sections 7.6 and 7.7.

Figure 7-15: Typical Cross Sections for Reverse-Flow Operation



(a) Cross Section at Underpass



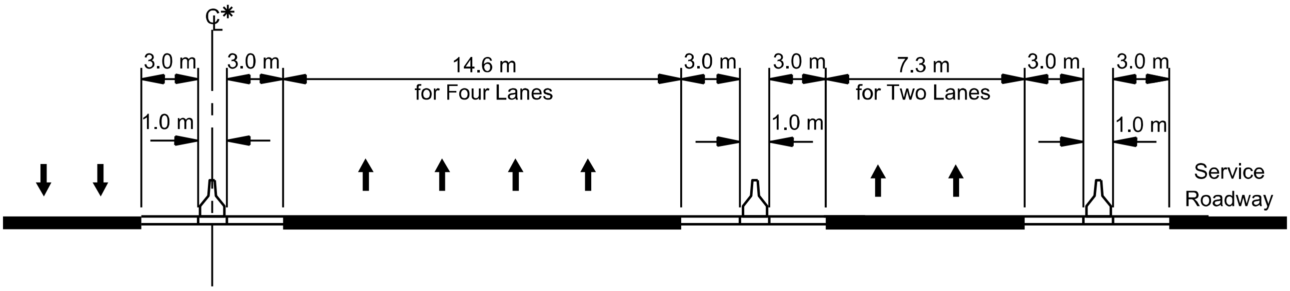
(b) Normal Section of Reversible Roadway

Source: (1)

7.3.5.2 Dual-Divided Freeways

Where more than a 4+4 design is required and the directional distribution of traffic is sufficiently balanced so that a reversible roadway is not appropriate, a dual-divided freeway made up of two-one-way carriageways in each direction of travel may provide the optimum facility. See Figure 7-16 and Figure 7-17. All four roadways are within the control-of-access lines. This type of cross section is sometimes referred to as "dual-dual." The outer freeway roadways usually serve the local interchange traffic, but they may also serve a substantial portion of the through traffic. For example, all trucks might be required to use the outer roadways only. Various arrangements are possible, depending on the character of traffic and crossroad conditions.

Figure 7-16: Typical Cross Sections for Dual-Divided Operation



* Roadway is symetrical about the centreline.

Figure 7-17: Dual-Divided Freeway

Dual-divided freeways usually function smoothly and carry extremely high volumes of traffic efficiently. Motorists using the inner roadways are removed from the weaving movement at frequently spaced interchanges, and disabled vehicles in either the inner or the outer roadway can quickly be moved to a shoulder.

Where the future need can be anticipated and sufficient right-of-way can be reserved, it may be practical to develop a dual-divided facility in two stages. Dual-divided construction may be the most practical solution to widening an existing freeway where the present traffic volumes are so great that the disruption in traffic during complete reconstruction cannot be tolerated.

Dual-divided facilities have great flexibility in their operation and maintenance. For example, during maintenance or reconstruction operations, one of the directional roadways may be temporarily closed during off-peak hours. Crash potential is greatly reduced by eliminating traffic conflicts with construction or maintenance work. Another advantage of dual-divided design is that the affected roadway can also be closed in case of a crash or other emergency, thereby facilitating clean-up operations.

Disadvantages of dual-divided facilities are the wide expanse of pavement and heavy traffic volumes that may disrupt an established community and tend to limit the continuity of the surrounding area. The dual-roadway system reduces the flexibility of traffic distribution, resulting in uneven distribution among lanes. The costs for right-of-way, construction, and maintenance may be greater than those on a normal divided facility with an equal number of lanes.

7.3.5.3 Freeways with Collector-Distributor Roadways

An arrangement having cross-sectional elements similar to the dual-divided freeway is a freeway with a collector-distributor (C-D) roadway system. The purpose of a C-D road is to eliminate weaving on the mainline freeway lanes and reduce the number of entrance and exit points on the through roadways while satisfying the demand for access to and from the freeway. C-D roadways may be provided within a single interchange (as discussed in Section 12.5.2), through two adjacent interchanges, or continuously through several interchanges of a freeway segment. See Figure 7-18 and Figure 7-19. Continuous C-D roads are similar to continuous service roads except that access to abutting property is not permitted.

The inside lanes are for the high-speed through traffic and the outside slower speed roadways are identified as C-D roads. Usually, the traffic volumes on the C-D system are less than those encountered on the dual-divided freeway, with fewer lanes. The minimum lane arrangement for a C-D system is two C-D lanes, 2+2 dual carriageway, and two C-D lanes. Other combinations may be developed as capacity needs warrant. Continuous C-D roadways should be integrated into a basic lane design to develop an overall system. Conduct a capacity analysis and basic lane determination for the overall system rather than just for the separate roadways.

Connections between the through roadways and C-D roadways are called “transfer roads.” Transfer roads may be either one or two lanes in width. The principle of lane balance applies to the design of transfer roads on both the through and C-D roadways. Both transfer and C-D roads should have shoulders equal in width to those of the through roadway. The outer separation should be as wide as practical with an appropriate barrier. Terminals of transfer roads should be designed in accordance with guidelines for ramp terminals, as presented in Section 12.6.

The design speed of C-D roadways is usually less than that of the through roadway because most of the turbulence caused by weaving occurs on the C-D roadways. A reduction in design speed of no more than 20 km/h is preferable for continuous C-D roadway systems.

Figure 7-18: Abu Dhabi Freeway with C-D Roadway

Figure 7-19: Dubai Freeway with C-D Roadways

7.4 General Design Considerations

7.4.1 Design Speed

As a general consideration, the design speed should be consistent with the anticipated operating speed of the freeway during both peak and non-peak hours, but the design speed should not be so high as to exceed the limits of prudent construction, right-of-way, and socioeconomic costs. For rural freeways, the design speed should be 120 km/h to 140 km/h and, for urban freeways, 100 km/h to 120 km/h. In mountainous terrain, a design speed of 80 km/h to 100 km/h is consistent with driver expectancy.

On many urban freeways, particularly in developing areas, a design speed of 100 km/h or higher can be provided with little additional cost. Where the freeway corridor is relatively straight, the character of the roadway and location of interchanges may be consistent with a higher design speed. Under these conditions, a design speed 120 km/h is desirable, because higher design speeds are closely related to the overall quality of a facility.

7.4.2 Design Traffic Volumes

Both rural and urban freeways should normally be designed to accommodate traffic projections for a 20-year period into the future, particularly for new construction. However, some elements of freeway reconstruction may be based on a shorter design period. Specific capacity needs should be determined from directional design hourly volumes (DDHV) for the appropriate design period. In large metropolitan areas, the selection of appropriate design traffic volumes and design periods may be influenced by system planning. Segments of freeways may be constructed or reconstructed to be commensurate with either intermediate traffic demands or with traffic based on the completed system, whichever may be more appropriate.

7.4.3 Capacity Analysis

Procedures for traffic operational analyses for freeways, including appropriate adjustments for operational and roadway factors, are presented in the *Highway Capacity Manual 2010* (17).

7.4.3.1 Freeway Segments

Freeways are composed of the following segments that are analysed to determine the overall capacity and level of service (LOS) freeway:

1. Freeway Merge and Diverge Segments. These are segments in which two or more traffic streams combine to form a single traffic stream (merge) (entrance ramps) or where a single traffic stream divides to form two or more separate traffic streams (diverge) (exit ramps). Section 12.6.3 provides guidance on capacity analyses for merge and diverge segments.

2. Freeway Weaving Segments. These are segments in which two or more traffic streams travelling in the same general direction cross paths along a significant length of freeway without the aid of traffic control devices (except for guide signs). Weaving segments are formed where a diverge segment closely follows a merge segment or where a one-lane exit ramp closely follows a one-lane entrance ramp and the two are connected by a continuous auxiliary lane. Section 12.3.13 provides guidance on capacity analyses for weaving segments.

3. Basic Freeway Segments. These segments are between the merge, diverge, or weaving segments.

7.4.3.2 Influence Areas

It is important that the definition of freeway segments and their influence areas be clearly understood. The influence areas of merge, diverge, and weaving segments are as follows:

1. Weaving Segment. This influence area includes the base length of the weaving segment plus 150 m upstream of the entry point to the weaving segment and 150 m downstream of the exit point from the weaving segment; entry and exit points are defined as the points where the appropriate edges of the merging and diverging lanes meet. See Figure 12-7.

2. Merge Segment. This influence area is measure from the point where the edges of the travel lanes of the merging roadways meet to a point 450 m downstream from that point. See Figure 12-7.

3. Diverging Segment. This influence area is measure from the point where the edges of the travel lanes of the merging roadways meet to a point 450 m upstream from that point. See Figure 12-7

7.4.3.3 Base Capacity

The base capacity represents the capacity of the facility, assuming that there are no heavy vehicles in the traffic stream and that all drivers are regular users of the segment. The base capacity for all freeway segments varies with the free-flow speed. See Figure 7-20 shows a sample speed-flow curves for a basic freeway section in the United States. The base lane capacity in Abu Dhabi for a freeway segment is assumed 2200 pc/h/ln. Note that this is less than the base capacity of 2400 pc/h/ln provided in the *Highway Capacity Manual* (17). To determine the actual lane capacity, use base capacity 2200 pc/h/ln and the methodologies as presented in the *Highway Capacity Manual 2010* (17).

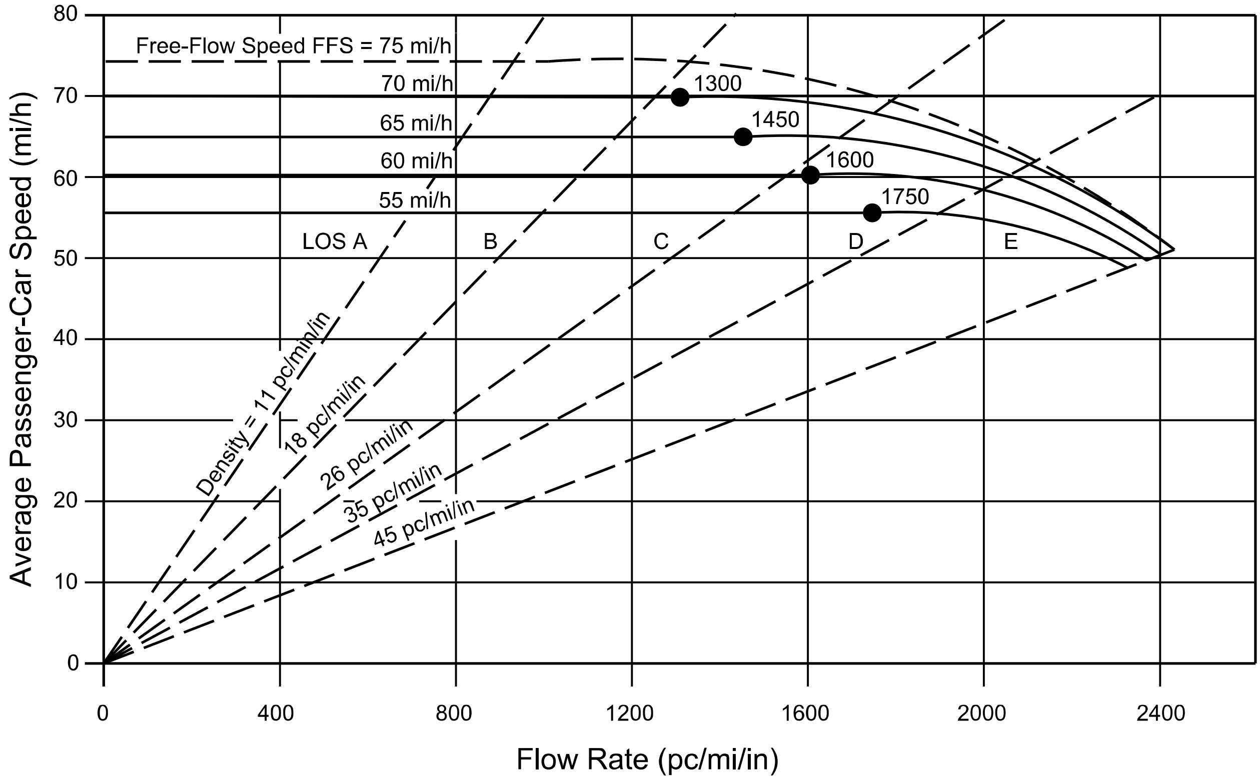
7.4.3.4 Level of Service

Because LOS for basic, weaving, merge, and diverge segments on a freeway is defined in terms of density, LOS for a freeway facility is also defined based on density. A facility analysis will result in a density determination and LOS for each component segment. The facility LOS will be based on the weighted average density for all segments with the defined facility. Weighting is done based on segment length and the number of lanes in each segment.

The LOS densities for a freeway facility and basic freeway segments are shown in Table 7-1. Guidance for merge and diverge segments, weaving areas, and ramps is provided in Chapter 12 “Interchanges.”

Freeways should generally be designed so that acceptable levels of congestion are not exceeded; desirably LOS B, with minimum LOS C. Freeway auxiliary facilities (i.e. ramps, mainline weaving sections, collector-distributor (C-D) roads) may be design with a LOS one level lower than the freeway segment if the overall freeway still meets the design LOS. In heavily developed sections of metropolitan areas, achieving level of service C may not be practical and the use of level of service D may be appropriate.

Figure 7-20: Sample Speed-Flow Curves for Basic Freeway Section



Source: (36)

Table 7-1: LOS Criteria for Freeway Facilities

LOS	Density (PC/mi/ln)
A	≤ 11
B	> 11 – 18
C	> 18 – 26
D	> 26 – 35
E	> 35 – 45
F	> 45 or any component v_d /c ratio > 1.00

Note: 1 Mile = 1.6 Km *Sources:* (17) / (36)

7.4.4 Travelled Way and Shoulders

The minimum freeway design should be a 2+2 dual carriageway. Through-traffic lanes should be 3.65 m wide. Freeway roadways should have a paved surface with adequate skid resistance and structural capacity. Pavement cross slopes should be 2% sloping away from the median. Guidance for ramp travelled-way widths are provided in Section 12.5.6.

Paved shoulders should be continuous on both the right and left sides of all freeway facilities. Shoulder widths on freeways are as follows:

1. 2+2. The median (or left) shoulder is normally 1.8 m wide (only if the ultimate design is a 2+2 freeway), at least 1.2 m of which should be paved and the remainder stabilized. The paved width of the right shoulder should be at least 3.0 m. Where the DDHV for truck traffic exceeds 250 veh/h, provide a paved right shoulder width of 3.65 m.
2. 3+3 or More. The paved width of the right and left shoulder should be 3.0 m. Where the DDHV for truck traffic exceeds 250 veh/h, provide a paved right shoulder width of 3.65 m.
3. Ramps. Guidance for ramp shoulder widths is provided in Section 12.5.6. Ramp shoulder widths are usually provided adjacent to acceleration and deceleration lanes with transitions to the freeway shoulder width at the taper ends.

7.4.5 Curbs

Caution should be exercised in the use of curbs on freeways. Where curbs are provided in special cases, they should not be closer to the travelled way than the outer edge of shoulder and should be easily traversable. An example of a special case in which shoulder curbs are used on freeways is at locations where curbs are provided to control drainage and reduce erosion. For more information, refer to the discussion on curb types and their placement in Section 3.2.6 and the Abu Dhabi *Roadside Design Guide* (28).

7.4.6 Superelevation

The desirable superelevation rate on freeways is 4%, with a maximum rate of 6%. See Section 5.3 for further guidance.

7.4.7 Grades

Maximum grades for freeways are presented in Table 7-2 for combinations of design speed and terrain type. Grades on urban freeways should be comparable to those on rural freeways of the same design speed. Steeper grades may be tolerated in urban areas, but the closer spacing of interchange facilities and the need for frequent speed changes make it desirable to use flatter grades wherever practical. On sustained upgrades, the need for climbing lanes should be investigated, as discussed in Section 6.3.

Table 7-2: Maximum Grades for Rural and Urban Freeways

Type of Terrain	Design Speeds (km/h)						
	80	90	100	110	120	130	140*
	Grades (%)*						
Level	4	4	3	3	3	3	3
Rolling	5	5	4	4	4	4	4
Mountainous	6	6	6	5	—	—	—

* *Grades 1% steeper than the value shown may be provided in urban areas with right-of-way constraints or where needed in mountainous terrain.*

* *Estimated by DMT.* *Source:* (1)

7.4.8 Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the principles in the Abu Dhabi *Road Structures Design Manual* (29).

The clear width on bridges carrying freeway traffic should be as wide as the approach roadway. On bridges longer than 60 m, some economy in substructure costs may be gained by building a single structure rather than twin parallel structures. In such cases, the approach shoulder widths are provided and a median barrier is extended across the bridge.

Structures carrying ramps should provide a clear width equal to the ramp width and paved shoulders.

The structure width and lateral clearance of roadways overpassing or underpassing the freeway are dependent on the roadway classifications as discussed in Chapter 8 “Rural Roads” and Chapter 9 “Urban Streets.”

7.4.9 Vertical Clearance

The vertical clearance to roadway structures, sign trusses, and pedestrian overpasses over freeways should be at least 6.5 m over the entire roadway width, including auxiliary lanes and the usable width of shoulders with consideration for future resurfacing. Where the freeway passes over a railroad, provide a vertical clearance of 7.5 m.

7.4.10 Roadside Design

Urban freeways at ground level and rural freeways should have clear zone widths consistent with their operating speed, traffic volume, and side slopes. Detailed discussions of clear zones and lateral offsets are discussed in the Abu Dhabi *Roadside Design Guide* (28).

Depressed freeways in urban areas have more restrictive rights-of-way, which may require retaining walls or bridge piers to be placed within the clear zone. Retaining walls and piers should not be located on the shoulder and preferably should be at least 0.6 m beyond the outer edge of shoulder. Retaining walls and pier crash walls should incorporate an integral concrete barrier shape, or offset from the shoulder to permit shielding with a separate barrier. Where walls are located beyond the clear zone or are not required, back slopes should be traversable and fixed objects within the clear zone should be of a breakaway design or shielded.

Elevated freeways on embankments generally warrant roadside barriers where slopes are steeper than 1V:3H or where the area beyond the toe of slope that remains within the clear zone is not traversable. The tops of retaining walls used in conjunction with embankment sections should be located no closer to the roadway than the outer edge of the shoulder, and the walls should incorporate the concrete barrier shape or be appropriately shielded.

7.4.11 Ramps and Terminals

The design of ramps and connections for all freeway types is covered in Section 12.5 and Section 12.6, respectively.

7.4.12 Outer Separations and Borders

An outer separation is defined as the area between the travelled way of the main lanes and a service (frontage) road or local street. A border is defined as the area between the service road or local street and the private development along the road. Where there are no service roads or local streets functioning as service roads, the area between the travelled way of the main lanes and the right-of-way limit should be referred to as the border. Because of the dense development along urban freeways, service roads are often necessary to maintain local service and to collect and distribute ramp traffic entering and leaving the freeway. Where the freeway occupies a full block, the adjacent parallel streets are usually retained as service roads.

The outer separation or border provides space for shoulders, verges, side slopes, drainage, access-control fencing, service areas, and in urban areas, retaining walls and ramps. In sensitive areas, the outer separation or border may also provide space for noise abatement measures. Usually, the outer separation is the most flexible element of an urban freeway section. Adjustment in width of right-of-way, as may be needed through developed areas, ordinarily is made by varying the width of the outer separation.

The outer separation or border should be as wide as economically practical to provide a buffer zone between the freeway and its adjacent area. The border should extend beyond the construction limits, where practical, to facilitate maintenance operations and encourage an effective roadside design. Wide outer separations also permit well-designed ramps between the freeway and the service road. The typical range in widths of outer separations is 25 m to 50 m, but much narrower widths may be used in urban areas if retaining walls are employed.

7.4.13 Access Control

A freeway should have no disruptions or traffic stopped at crossroads anywhere along the route. All opposing traffic movements are separated by physical constraints (e.g. grade separations, non-traversable median separators). Access to the freeway is by directional ramps. All ramps should be suitably spaced and designed to provide the minimum differential between the speed of the through traffic stream and the speed of the merging or diverging vehicles.

Private direct access to a freeway is prohibited. Abutting property must access the freeway via a secondary roadway network. Any direct access for construction activity must be within the authorised project construction zone and in accordance with approved traffic control plans.

For interchanges in rural areas, a spacing of 8 km is preferred, with an interchange minimum spacing distance of 4 km. See the Abu Dhabi *Access Management Policy and Procedures Manual* (21) for further guidance. The separation between on on-ramps and the next downstream off-ramp is the most critical distance in consideration of spacing. Interchange centreline spacing criteria cannot anticipate how topography and other conditions may result in ramps too closely spaced. A traditional cloverleaf is undesirable as the ramp activity overlaps. On rural roadways, there should be over 2000 m between gore points.

A minimum spacing of 2 km between interchanges is preferred in urban areas. Where there is a heavy on-ramp volume followed by a high off-ramp volume, a greater spacing is desirable. Sequential off-ramps can be closer together, but not for sequential on-ramps. Where there is a high frequency of trucks and buses, acceleration lanes should be twice the normal distance.

Right-turn-only ramps are allowed when there is a necessity for a low-volume isolated access (e.g. official utility installation) or traveller service area (e.g. gas station, rest area) directly related to the right-of-way use or travel support.

Parking is not permitted along freeway roadways or in the roadside area.

Official stopping areas including rest stops, filling stations with public facilities and shops must be outside the roadway limits (i.e. beyond the roadway right of way). The ramps for these facilities must be spaced and designed in the same manner as interchange ramps; see Section 12.3.3. Where practical, public facilities should be on both sides of the road; otherwise, there may be incidents of vehicles crossing the median. Design the freeway median to discourage crossover activities.

Public facilities for public transport stops, pedestrian crossings, footways, and cycle lanes must be grade separated or there must be a physical barrier between the freeway roadway and the other public facility.

Within the general corridor area of the freeway, there should always be a parallel secondary route for non-freeway travel. This can be in the form of service roads and other secondary classifications. In extremely low population areas, secondary systems may be isolated ending at freeway access points; see Figure 7-21.

Figure 7-21: Isolated Freeway Access

7.5 Grade Separation Design

On fully access-controlled facilities, each intersecting roadway must be terminated, rerouted, or provided with a grade separation or interchange. The importance of the continuity of the crossing road, the feasibility of alternative routes, traffic volumes, construction costs, environmental impacts, etc., must be evaluated to determine the option that is the most cost effective.

7.5.1 Justification

Section 12.2 provides guidelines that must be considered in determining whether an interchange should be provided. In general, interchanges are provided at all freeway-to-freeway crossings and at other roadways based on the anticipated demand for access.

For each crossroad along the freeway, which is not an interchange, a determination must be made whether the crossroad should be closed, rerouted, or provided with a grade separation. This justification is made primarily by comparing the respective cost and social factors for each alternative. Figure 7-22 shows a grade separated crossing. Although cost is a primary factor, also consider the following:

Figure 7-22: Grade Separated Freeway Crossing

1. Operations. Grade separations should be of sufficient number and adequate capacity to accommodate crossroad traffic, traffic diverted to crossroads from other roads and streets terminated by the freeway, and the traffic generated by access connections to and from the mainline.

2. Rural/Urban Locations. In rural areas, the location of grade separation structures is determined by an access and feasibility study. For urban areas, usually grade separation structures are provided every three to four blocks for continuity.

3. Local Considerations. Closing the crossroad can have a significant effect on local users and the overall local road system integrity, due primarily to changes in travel patterns. These may include:

a. Emergency Personnel. The financial effect of the longer detour route on emergency vehicles is generally not a concern. However, the extra response time could adversely affect the health and safety of local citizens.

b. Businesses. Evaluate access to businesses to ensure that these operations can continue without severe economic hardship. Road closures can significantly affect their deliveries and the number of customers they receive (e.g. customers may be unwilling to travel the extra distance).

c. Social Factors. Parks, mosques, cemeteries, public facilities, and other areas or buildings of social concern generally cannot be relocated. Limited access to these facilities may create undue hardship if a specific road is closed.

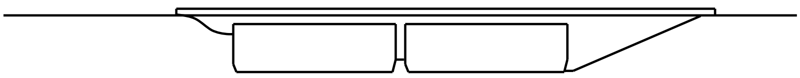
d. Land Use Planning. Consider future land use within an urban environment to ensure adequate access and reciprocation factors are available.

7.5.2 Design

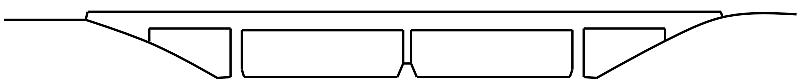
7.5.2.1 Structure Type

Varieties of structures are used for grade separation of intersections highways. Similar to an overcrossing roadway, the full roadway approach width should be carried through the undercrossing, including roadside barriers. Figure 7-23 illustrates several variations for carrying the freeway cross section through the underpass:

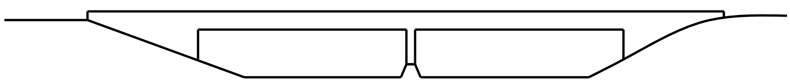
Figure 7-23: Typical Cross Sections for Underpasses



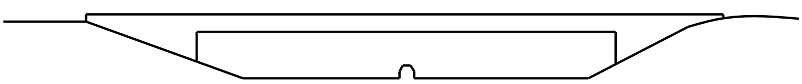
(a)



(b)



(c)



Source: (36)

(d)

- Figure 7-23(a) shows a bridge with solid abutments. The left side fits a walled section on the roadway approach; the right side intercepts the side slope.
- Figure 7-23(b) shows a structure with open-end spans.
- Figure 7-23(c) illustrates where the side slopes of partial height pass through the structures. This design provides the approaching driver a greater degree of openness with la less constricting and safer roadside.
- Figure 7-23(d) is an extension of Figure 7-23(c) where the entire cross section of the freeway is spanned. A greater sense of openness is produce.

7.5.2.2 Over Versus Under

A thorough study should be completed for each proposed roadway grade separation to conclude whether the major roadway should be over or under the other road(s). The main issues that dictate whether a road should be carried over or under usually fall into one of the following:

- topography predominates, which should lead to a design that is fitted to it;
- the topography does not help one design; and
- the grade-line and alignment of one roadway predominate, and so the final design should accommodate that roadway's alignment instead of the site topography.

Figure 7-24 illustrates roundabout interchanges with the freeway above and below the roundabout.

Figure 7-24: Roundabout Interchanges with Freeway Over and Under

Typically, a design that is in line with the existing topography is also the most economical and aesthetically pleasing. This factor should usually be considered first. Otherwise, it is appropriate to study the following secondary factors:

1. <u>The Whole Interchange</u>. For the most part, designers are governed by the need for economy, which is obtained by designs that fit existing topography, not only along the intersecting highways but also for the whole of the area to be used for ramps and slopes. Thus, it is

appropriate to consider alternatives in the interchange area as a whole to decide whether the major road should go over or under the crossroad.

2. Approach View. An undercrossing highway has a general advantage in that an approaching interchange may be easily seen by drivers. As a driver approaches, the structure appears ahead, making the presence of the upper-level crossroad obvious, and providing advance warning of the likely presence of interchange ramps.

3. Aesthetics. Through traffic is given aesthetic preference by a layout in which the more important road is the overpass. A wide overlook can be provided from the structure and its approaches, giving drivers a minimum feeling of restriction.

4. Turning Traffic. Where turning traffic is significant, the ramp profiles are best fitted when the major road is at the lower level. The ramp grades then assist turning vehicles to decelerate as they leave the major roadway and to accelerate as they approach it, rather than the reverse. In addition, for diamond interchanges the ramp terminal is visible to drivers as they leave the major roadway.

5. Terrain. In rolling topography or in rugged terrain, major-road overcrossings may be attainable only by a forced alignment and rolling gradeline. Where there otherwise is no pronounced advantage to the selection of an underpass or overpass, the design that provides the better sight distance on the major road should be preferred.

6. Stage Construction. An overpass offers the best possibility for stage construction, both in the roadway and structure, with minimum impairment of the original investment. The initial development of only part of the ultimate width is a complete structure and roadway in itself. By lateral extension of both or construction of a separate structure and roadway for a dual carriageway, the ultimate development is reached without loss of the initial facility.

7. Drainage. Drainage challenges may be reduced by carrying the major roadway over the crossroad without altering the crossroad grade. In some cases, the drainage concerns alone may be sufficient reason for choosing to carry the major roadway over rather than under the crossroad.

8. Economics. The cost of bridges and approaches may determine whether the major roadway underpasses or overpasses the minor facility. A cost analysis that takes into account the bridge type, span length, roadway cross section, angle of skew, soil conditions, and cost of approaches will determine which of the two intersecting roadways should be placed on structure.

9. Earthwork. An underpass may be more advantageous where the major road can be built close to the existing ground, with continuous gradient and with no pronounced grade changes. Where the widths of the roads differ greatly, the quantity of earthwork makes this arrangement more economical. Because the minor road usually is built to less generous design criteria than the major road, grades on it may be steeper and sight distances shorter, with resultant economy in grading volume and pavement area on the shorter length of road to be rebuilt above the general level of the surrounding country.

10. Freeway System. Frequently, the choice of an underpass at a particular location is determined not by conditions at that location, but by the design of the freeway as a whole.

Grade separations near urban areas constructed as parts of a depressed freeway, or as one raised above the general level of adjoining streets, are good examples of cases where decisions regarding individual grade separations are subordinated to the general development.

11. Existing Freeway. Where a new roadway crosses an existing route carrying a large volume of traffic, an overcrossing by the new road causes fewer disturbances to the existing route and a detour is usually not needed.

12. Vertical Clearance. The overcrossing structure has no limitation as to vertical clearance, which can be a significant advantage in the case of oversized loads requiring special permits on major routes.

13. Traffic Volumes. Desirably, the roadway carrying the highest traffic volume should have the fewest number of bridges for better rideability and fewer conflicts when repair and reconstruction are required.

14. Noise. In some instances, it may be appropriate to have the higher volume facility depressed and crossing under the lower volume facility to reduce noise impact.

7.5.2.3 Distance to Obtain Grade Separation

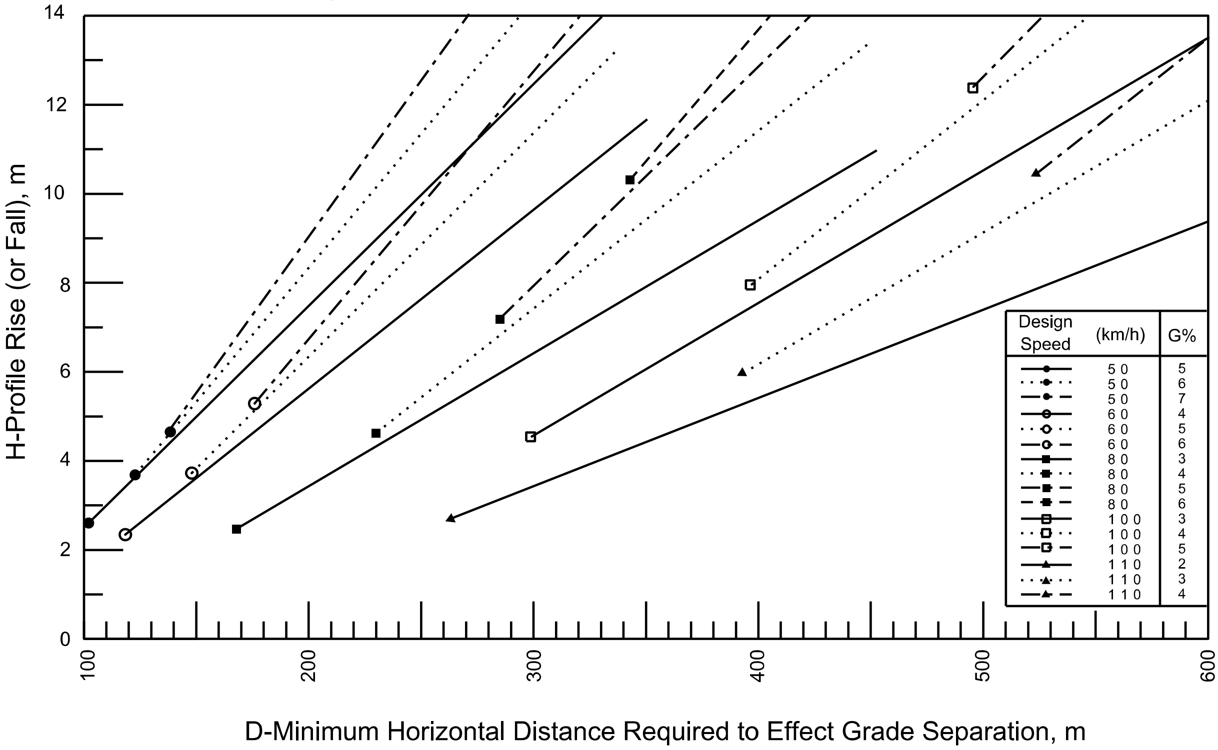
The distance required for adequate design of a grade separation depends on the design speed, the roadway gradient, and the amount of rise or fall necessary to affect the separation. Figure 7-25 can be used during the planning phase to quickly determine whether a grade separation is feasible for a given set of conditions, what gradients may be involved, and what profile adjustments may be necessary on the crossroad. In addition, the designer needs to carefully study the sight distance requirements because these will often dictate the required horizontal distance along the crossroad.

When using Figure 7-25 consider the following:

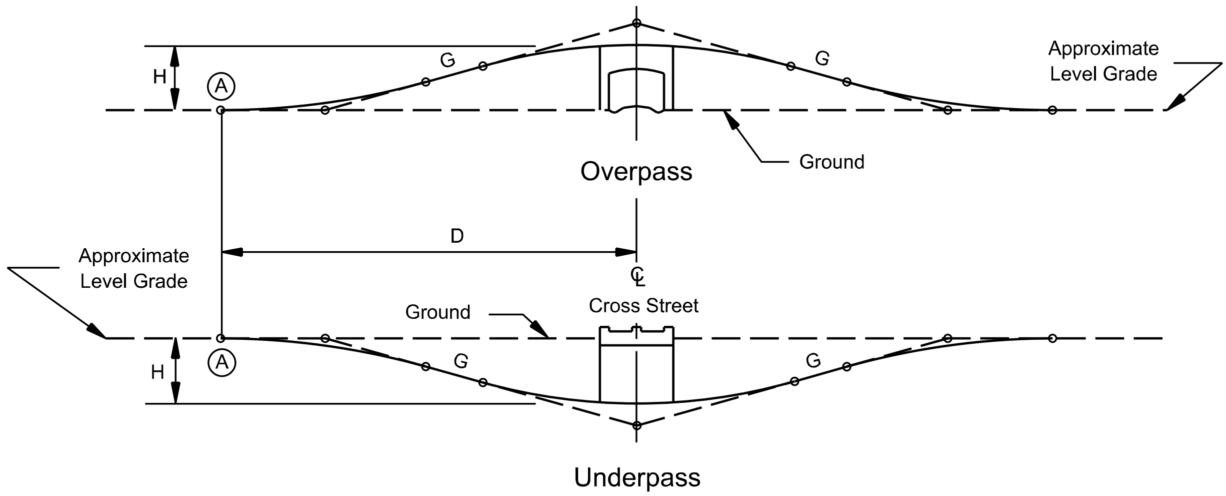
1. Minimum Horizontal Distances. The plotted lines on Figure 7-25 are derived assuming the same approach gradient on each side of the structure. However, values of “D” from the figure also are applicable to combinations of unequal gradients. Distance “D” is equal to the length of the initial vertical curve, plus one-half the central vertical curve, plus the length of tangent between the curves. Lengths of vertical curves are based on the stopping sight distance. However, longer vertical curves are desirable from an aesthetic and safety standpoint. Conversely, longer curve lengths may be costlier due to increased earthwork quantities. However, these additional costs may be a less important consideration if crossroads or access points exist near the grade separation structure.

2. Maximum Gradient. The lower terminal point of each gradient line on Figure 7-25, marked by a small symbol, indicates the distance where the tangent between the curves is zero and below which a design for the given grade is not feasible (i.e. a profile condition where the minimum central and end curves for the gradient would overlap).

Figure 7-25: Grade Separation Determination



Note: Symbols on each line indicate the point below which the grade is not feasible, necessitating the use of the next flatter grade.



Source: (1)

3. Restricted Gradients. For the usual profile rise or fall required for a grade separation ("H" of 7.5 m or more), do not use gradients greater than 3% for a design speed of 110 km/h or greater, 4% for 100 km/h, 5% for 80 km/h, and 6% for 60 km/h. For values of "H" less than 7.5 m, use flatter gradients.

4. Relationship. For a given "H" and design speed, distance "D" is only shortened a negligible amount by increasing the gradient. However, the distance "D" varies to a greater extent for a given "H" and "G" with respect to the design speed.

5. Elevation. Considering the vertical clearance and structural depth, an elevation distance of "H" is typically between 7.0 m and 7.5 m for the grade separation of two roadways. "H" is typically the same for a freeway under a railroad. For a railroad facility under a freeway, "H" is typically 9.0 m to 9.5 m.

7.6 Rural Freeways

Rural freeways are generally ground-level freeways with high design speed, flatter alignment, wider cross sectional elements and wider rights-of-way. Table 7-3 summarises the geometric design criteria for rural freeways. Figure 7-26 shows a rural freeway with lay-bys.

Rural freeways are initially designed to accommodate anticipated traffic growth for a 20-year period and to remain in service for a much longer time. Any cost savings that might potentially be gained by initially constructing for a lesser design period would likely be offset by the high costs, disruption to the environment, and inconvenience to traffic that would accompany later reconstruction of major facilities.

Rural freeways generally designed as 3+3 dual carriageway except on approaches to metropolitan areas where more lanes may be provided. Interchanges are typically provided where intersecting roadways are classified as collectors and higher. Local roads may be terminated at the freeway, connected to service roads or other local roads for continuity of travel, or carried over or under the freeway by grade separation with or without an interchange.

Figure 7-26: Rural Freeway with Lay-bys

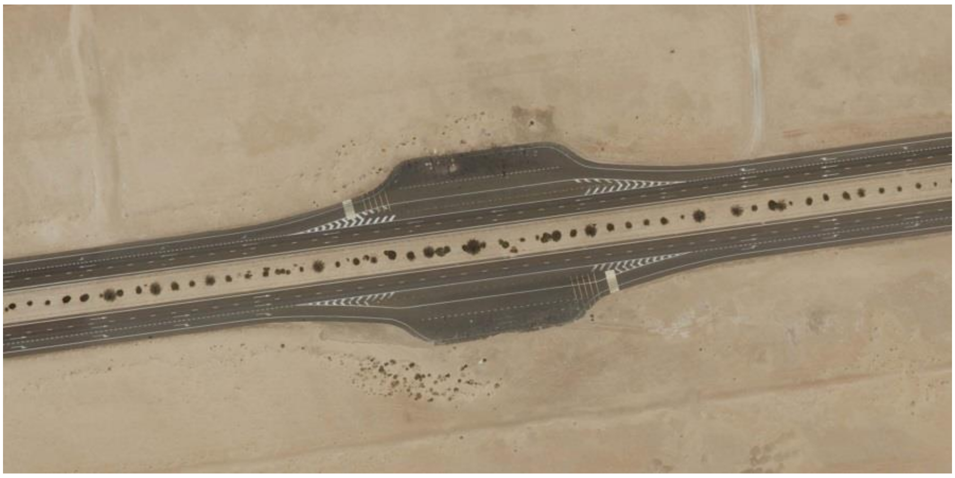


Table 7-3: Geometric Design Criteria — Rural Freeways

Design Element				Manual Section	Design Criteria
Design Controls	Design Year			2.7	20 Years
	Design Speed			7.4.1	Des.: 140 km/h Min.:120 km/h
	Access Control			7.4.13	Full Control
	Level of Service			7.4.3	Des: B Min: C
Cross Section Elements	Travelled Way	Lane Width		7.4.4	3.65 m
		Surface Type		7.4.4	Paved
	Shoulder Width	Right	Total Width **(1c)**	7.4.4	3.0 m
			Paved Width	7.4.4	3.0 m **(1a)**
			Surface Type	7.4.4	Paved
		Left	Total Width **(1c)**	7.4.4	1.2 m
			Paved Width	7.4.4	1.2 m **(1b)**
			Surface Type	7.4.4	Paved
	Auxiliary Lanes	Lane Width		7.4.4	3.65 m
		Shoulder Width		7.4.4	Desirable: Full Shoulder Width Minimum: 1.8 m
	Cross Slope	Travel Lane		7.4.4	2% Typical
		Shoulder		7.4.4	2% Typical
		Auxiliary Lanes		7.4.4	2% Typical
	Median Width **(2)**	Depressed		7.6.2	Recommend: 10 m Desirable: 18 m
		Flush (Concrete Barrier)		7.6.2	Minimum: 7.8 m
	Right-of-Way Width			3.6	140 m **(3)**
Roadway Slopes	Side Slopes	Cut Section **(4)**	Front Slope	3.4	1:6
			Ditch Width		2.4 m
			Back Slope		1:4
		Rock Cut		3.4	See Section 3.4
		Fill Section	Height: 0-1 m	3.4	1:6
			Height: 1-3 m		1:6 to Clear Zone; 1:4 to Toe
			Height: >3 m		1:6 to Clear Zone; 1:3 to Toe
	Verges			3.4	1.0 m – 1.2 m
Bridges	Clear Roadway Width			7.4.8	Full Approach Roadway Width
	Vertical Clearance (Freeway Under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)			7.4.9	6.5 m
	Vertical Clearance (Freeway over Railroad)			7.4.9	7.5 m

Design Element			Manual Section	Design Speed 140 km/h	Design Speed 130 km/h	Design Speed 120 km/h
Alignment	Stopping Sight Distance (Level)		4.3	325 m	285 m	250 m
	Decision Sight Distance		4.4	400 m	390 m	360 m
	Minimum Radii	e_{max} = 0.04	5.2	1403 m	1109 m	872 m
		e_{max} = 0.06	5.2	1188 m	957 m	756 m
	Vertical Curvature (K-values)	Crest	6.3	161	124	95
		Sag	6.3	84	73	63
	Maximum Grade (**5**)	Level	7.4.7	3%	3%	3%
		Rolling	7.4.7	4%	4%	4%
		Mountainous	7.4.7	—	—	—
	Minimum Grade		7.4.7	Desirable: 0.5% Minimum: 0%		

1. Shoulder Width. The following will apply:

   a. Barriers. All shoulder widths should desirably be increased by 0.6 m when a barrier is present.

2. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes), and field conditions.

3. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths, plus the outside shoulder widths, plus the median width, plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus services corridor.

4. Cut Slopes. Typical values in table apply to earth cuts; see Section 3.4 for rock cuts.

5. Maximum Grade. With wide medians where two roadways are on independent alignments, downgrades may be 1% steeper. Freeway grades in mountainous terrain may be 1% steeper.

7.6.1 Alignment and Profile

Rural freeways generally have high volumes and high speeds. They should have smooth flowing horizontal and vertical alignments with appropriate combinations of flat curvature and gentle grades. Even though the profile may satisfy all the design controls if minimum criteria are used, the finished vertical alignment may appear forced and angular. The designer should check profile designs in long continuous plots to help avoid an undesirable roller-coaster alignment in rolling terrain. The relation of horizontal and vertical alignment should be studied simultaneously to obtain a desirable combination. See Chapter 5 “Horizontal Alignment” and Chapter 6 “Vertical Alignment” for further guidance.

Advantage should be taken of favourable topographic conditions to incorporate variable median widths and independent roadway alignments to enhance the aesthetic aspects of freeways. Avoid changing median widths on tangent alignments, where practical, so as not to introduce a distorted appearance.

Because there are usually fewer physical constraints in constructing the rural road network than its urban counterpart, rural freeways can usually be constructed near ground level with smooth and relatively flat profiles. The profile of a rural freeway is controlled more by drainage and earthwork considerations and less by the need for frequent grade separations and interchanges.

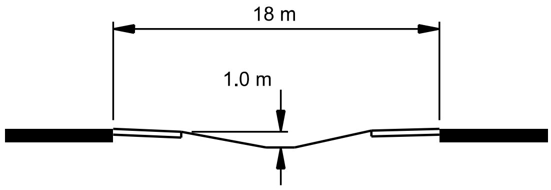
7.6.2 Medians

Median width is defined as the dimension between edges of the travelled way for the roadways in opposing directions of travel, including the width of the left shoulders, if any. Median widths of 15 m to 30 m are common on rural freeways. The 18-m dimension shown in Figure 7-27(a) provides for 3.0-m graded shoulders and 1V:6H front slopes with a 1.0-m median ditch depth. Adequate space is provided for vehicle recovery; however, median piers may need shielding in accordance with the Abu Dhabi *Roadside Design Guide* (28). The 30-m dimension shown in Figure 7-27(b) permits the designer to use independent profiles in rolling terrain to blend the freeway more appropriately within the environment while maintaining flat slopes for vehicle recovery. In flat terrain, the 30-m median is also suitable when stage construction will add two 3.65-m travel lanes in the future.

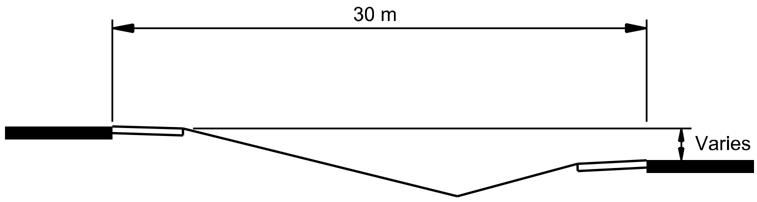
Where the terrain is extremely rolling or the land is not suitable for cultivation or grazing, a wide variable median with an average width of 45 m or more, as shown in Figure 7-27(c), may be attainable. This width permits the use of independent roadway alignment, both horizontally and vertically, to its best advantage in blending the freeway into the natural topography. The wider independent roadways allow medians to be left in their natural state of vegetation, trees, and rock outcroppings to reduce maintenance costs and add scenic interest to passing motorists. The combination of independent alignment and a natural park-like median is pleasing to motorist. However, for driver reassurance, the opposing roadway should be in view at frequent intervals.

Median widths in the range of 3.0 m to 9.0 m, as shown in Figure 7-27(d), may be needed where right-of-way restrictions dictate or in mountainous terrain. A median barrier will be required; see Abu Dhabi *Roadside Design Guide* (28) for guidance.

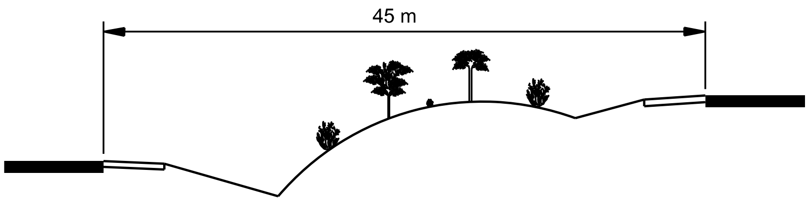
Figure 7-27: Typical Rural Medians



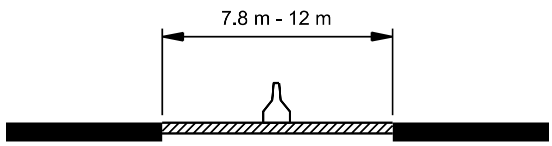
(a) Depressed, Parallel Alignment



(b) Depressed, Independent Profile



(c) Separate Roadways, Natural Median



(d) Paved Flush with Barrier

To avoid excessive adverse travel for emergency and law-enforcement vehicles, emergency crossovers on rural freeways may be provided where interchange spacing exceeds 8 km. Between interchanges, emergency crossovers may be spaced at 5-km to 6-km intervals or as needed. Maintenance crossovers may be needed at one or both ends of interchange facilities, depending on interchange type, to facilitate maintenance operations. Maintenance or emergency crossovers generally should not be located closer than 450 m from the end of a speed-change taper of a ramp or to any structure. Crossovers should be located only where above minimum stopping sight distance is provided, and preferably should not be located on superelevated curves.

The width of the crossover should be sufficient for turning movements and should have a surface capable of supporting maintenance equipment used on it. The crossover should be depressed below shoulder level to be inconspicuous to traffic and should have 1V:10H or flatter side slopes so that the median is traversable for vehicles that run off the road. Crossovers should not be placed in restricted-width medians unless the median width is sufficient to accommodate the appropriate design vehicle length. Where median barriers are employed, each end of the barrier at the median opening may need a crashworthy terminal. For further information, see the Abu Dhabi *Roadside Design Guide* (28).

7.6.3 Side Slopes

Flat, rounded side slopes, fitting with the topography and consistent with available right-of-way, should be provided on rural freeways. Front slopes of 1V:6H or flatter are recommended in cut sections and for fills of moderate height, as discussed in Section 3.4.2. Where fill heights are intermediate, a combination of recoverable and non-recoverable slopes may be used to provide the acceptable vehicle recovery area. For high fills, steeper slopes protected by guardrail may be needed. Where rock or loess deposits are encountered, back slopes may be nearly vertical, but, where practical, should be located to provide an adequate recovery area for errant vehicles. For additional side slope design information, see the Abu Dhabi *Roadside Design Guide* (28).

7.6.4 Service Roads

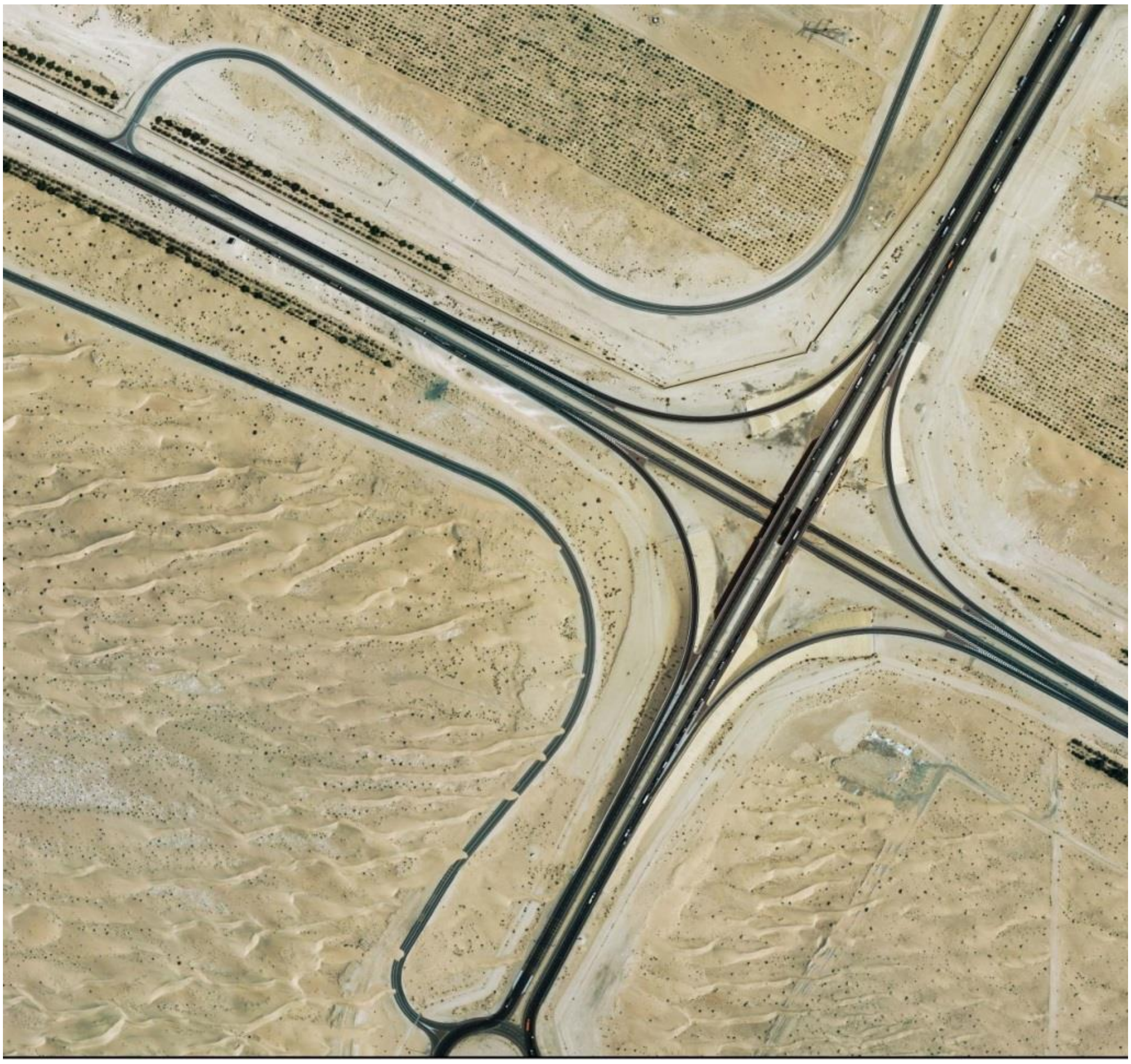
The need for local service across and along rural freeway corridors is usually considerably less than the need along highly developed urban freeways. Therefore, along rural freeways, service (frontage) roads are usually intermittent and relatively short. Service roads either provide access to one or more severed properties or provide continuity of a local road by connecting it with a grade-separated crossroad.

Because of the lack of continuity and irregularity of the adjoining street system and the type of service being provided, newly constructed rural service roads are typically two-way facilities. Where a rural freeway is located parallel to and in close proximity to a major roadway, the major roadway is often converted to a continuous two-way service road and should be designed as a rural collector.

Traffic operations are more complex at two-way service road intersections with grade-separated crossroads; therefore, locate these intersections as far as practical from grade-separation structures and interchange ramp terminals. Two-way service road intersections using stopped control should be located 120 m to 150 m from the interchange ramp terminals. In outer urban areas, intersections may be signal controlled because of traffic volume. In this case, service road intersections should be located 210 m to 300 m from the interchange ramp terminals. If the frontage road intersections with the crossroad are right-in/right-out only, then they can be located 60 m to 90 m from the interchange ramp terminals.

Rural service roads are generally outside the control-of-access line, but within the right-of-way limits. Section 7.4.12 provides guidance on the separation between the freeway and service roads. Design rural service roads using the design criteria for rural collectors or rural local roads, as discussed in Chapter 8 "Rural Roads." The design speed for a service road will be based on its functional classification (collector or local road) and not be more than 40 km/h less than the freeway design speed. Figure 7-28 illustrates service roads adjacent to a rural freeway.

Figure 7-28: Rural Freeway with Service Roads



7.7 Urban Freeways

7.7.1 General Design Characteristics

Urban freeways are capable of carrying high traffic volumes. While urban freeways may have from 4 to 16 through-traffic lanes, they are typically designed as 4+4 dual carriageways. Urban freeways are classified as depressed, elevated, ground-level, or combination-type. See Section 7.3 for a discussion on these various freeway types.

Table 7-4 summarises the general design criteria for urban freeways. Figure 7-29 shows a system interchange between two urban freeways.

Table 7-4: Geometric Design Criteria —Urban Freeways

	Design Element			Manual Section	Design Criteria		
Design Controls	Design Year			2.7	20 Years		
	Design Speed			7.4.1	Des.: 120 km/h Min.:100 km/h		
	Access Control			7.4.13	Full Control		
	Level of Service			7.4.3	Des: B Min: D		
Cross Section Elements	Travelled Way	Lane Width		7.4.4	3.65 m		
		Surface Type		7.4.4	Paved		
	Shoulder Width	Right	Total Width **(1c)**	7.4.4	3.0 m		
			Paved Width	7.4.4	3.0 m **(1a)**		
			Surface Type	7.4.4	Paved		
		Left	Total Width **(1c)**	7.4.4	1.2 m		
			Paved Width	7.4.4	1.2 m **(1b)**		
			Surface Type	7.4.4	Paved		
	Auxiliary Lanes	Lane Width		7.4.4	3.65 m		
		Shoulder Width		7.4.4	Desirable: Full Shoulder Width Minimum: 1.8 m		
	Cross Slope	Travel Lane		7.4.4	2% Typical		
		Shoulder		7.4.4	2% Typical		
		Auxiliary Lanes		7.4.4	2% Typical		
	Median Width **(2)**	Depressed		7.7.2	Recommend: 10 m Desirable: 18 m		
		Flush (Concrete Barrier)		7.7.2	Minimum: 7.8 m		
	Right-of-Way Width			3.6	140 m **(3)**		
Roadway Slopes	Side Slopes	Cut Section **(4)**	Front Slope	3.4	1:6		
			Ditch Width		2.4 m		
			Back Slope		1:4		
		Rock Cut		3.4	See Section 3.4		
		Fill Section	Height: 0-1 m	3.4	1:6		
			Height: 1-3 m		1:6 to Clear Zone; 1:4 to Toe		
			Height: > 3 m		1:6 to Clear Zone; 1:3 to Toe		
	Verges			3.4	1.0 m – 1.2 m		
Bridges	Clear Roadway Width			7.4.8	Full Approach Roadway Width		
	Vertical Clearance (Freeway Under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)			7.4.9	6.5 m		
	Vertical Clearance (Freeway over Railroad)			7.4.9	7.5 m		
Alignment					**Design Speed**		
					120 km/h	**110 km/h**	**100 km/h**
	Stopping Sight Distance (Level)			4.3	250 m	220 m	185 m
	Decision Sight Distance			4.4	470 m	430 m	400 m
	Minimum Radii		$e_{max} = 0.04$	5.2	872 m	635 m	492 m
			$e_{max} = 0.06$	5.2	756 m	561 m	438 m
	Vertical Curvature (K-values)		Crest	6.3	95	74	52
			Sag	6.3	63	55	45
	Maximum Grade (**5**)		Level	7.4.7	3%	3%	3%
			Rolling	7.4.7	4%	4%	4%
	Minimum Grade			7.4.7	Desirable: 0.5% Minimum: 0%		

1. Shoulder Width. The following will apply:

   a. Barriers. All shoulder widths should desirably be increased by 0.6 m when a barrier is present.

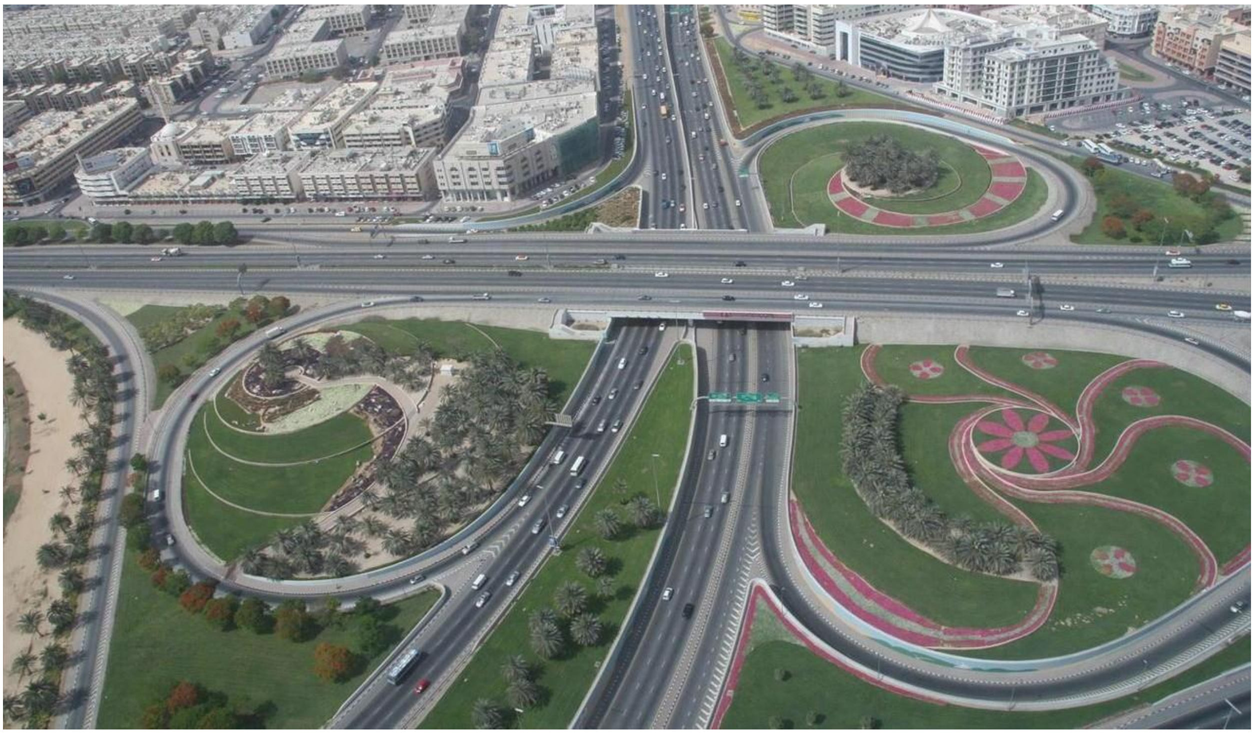
2. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e., planned addition of travel lanes), and field conditions.

3. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths, plus the outside shoulder widths, plus the median width, plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus service requirements. ROW width is applicable only if the minimal requirements of other agencies have been met (e.g. DMT, utilities, municipalities). DMT will determine the final ROW width.

4. Cut Slopes. Typical values in table apply to earth cuts; see Section 3.4 for rock cuts.

5. Maximum Grade. Grades 1% steeper may be used for restricted areas where development precludes the use of flatter grades.

Figure 7-29: Urban Freeways



7.7.2 Medians

A wider separation between traffic in opposing directions is more comfortable for motorists and can reduce the frequency of cross-median collisions involving vehicles that run off the road into the median; therefore, the median on urban freeways should be as wide and flat as practical. Additional median width may be used for mass transit or to provide additional lanes if more capacity is needed in the future. However, in densely developed areas with expensive-right-of-way, the width available for a median is usually restricted. The minimum median width for a 2+2 freeway should be 3.0 m, which provides for two 1.8-m shoulders and a 0.6-m median barrier. For 3+3 freeways or more lanes, the minimum median width should be 6.6 m. See the Abu Dhabi *Roadside Design Guide* (28) for guidance in determining the need for median barriers. Where a median barrier is used, additional lateral offset may be required to provide minimum stopping sight distance along the inside lane on sharper curves.

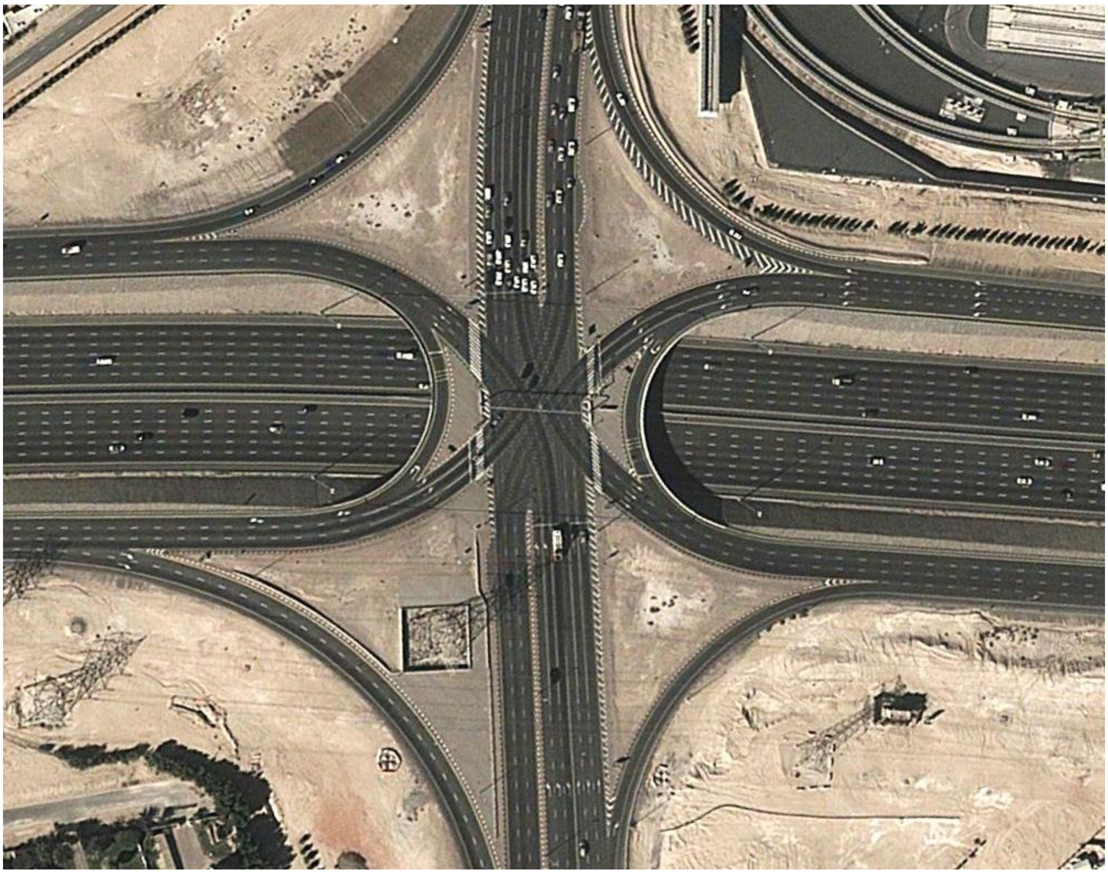
Median crossovers for emergency or maintenance purposes are not generally warranted on urban freeways due to the close spacing of interchange facilities and the extensive development of the abutting street network.

7.7.3 Service Roads

Along urban freeways, service (frontage) roads are usually provided on one or both sides of the freeway. Service roads either provide access to one or more severed properties or provide continuity of a local road by connecting it with a grade-separated crossroad.

From an operational and safety perspective, one-way service roads are preferred to two-way. One-way operations may inconvenience local traffic to some extent, but the advantages in reducing vehicular and pedestrian conflicts at intersecting streets often fully compensates for this inconvenience. In addition, the right-of-way required is reduced and potential for wrong-way conflicts is reduced significantly. Figure 7-30 illustrates one-way service roads at an interchange.

Figure 7-30: Single Point Diamond Interchange with One-Way Service Roads



Slip exit and entrance ramps may be used with one-way service roads. Exit and entrance ramps primarily serve the cross street with a secondary function of serving the service road. Between the freeway entrance and exit there may be weaving on the freeway. In addition, there is usually weaving on the service road, particularly between the slip ramp entrance onto the service road and the service road/cross-street intersection. This portion of the frontage road must be carefully designed in conjunction with its cross-street intersection for successful operation. Also important is the operation of the frontage road between the crossroad intersection and the diverge of the freeway entrance ramp from the frontage road. For detailed guidance on designing slip ramps with frontage roads, see Sections 12.4.4.2 and 12.7.2.

Two-way service roads at high-volume, urban intersections may complicate crossing and turning movements. Do not use off ramps (slip ramps) with two-way service roads because of the potential for wrong-way entry. The reduction in vehicular and pedestrian conflicts outweighs the operational inconveniences of the one-way operation.

Two-way service roads may be considered for partially developed urban areas where the adjoining street system is considered non-conventional and one-way service roads would provide extreme travel distance and cause undue inconvenience. Other appropriate situations for two-way service roads are where points of access to the through facility are low, where only one service road is provided, or where roadways connecting with the service roads are widely spaced. Two-way service roads should be considered where there is no parallel roadway within reasonable distance of the service road. Design urban service roads using the Streets classification as discussed in Chapter 9 “Urban Streets.”

7.7.4 Managed Lanes

7.7.4.1 General Considerations

Managed lanes are defined as roadway facilities or a set of lanes where operational strategies are implemented and managed in response to changing conditions to increase freeway efficiency, maximise capacity, manage demand, and generate revenue. Examples of managed lanes include high-occupancy vehicle (HOV) lanes, value-priced lanes, high-occupancy toll (HOT) lanes, and exclusive or special use lanes (e.g. express lanes, bus lanes, transit lanes, reversible flow lanes). For additional information on managed lanes, review the following United States Federal Highway Agency documents *Managed Lanes: A Cross Cutting Study* (37) and the *Freeway Management and Operations Handbook* (38) and the Institute of Transportation Engineer’s *Freeway and Interchange Geometric Design Handbook* (36).

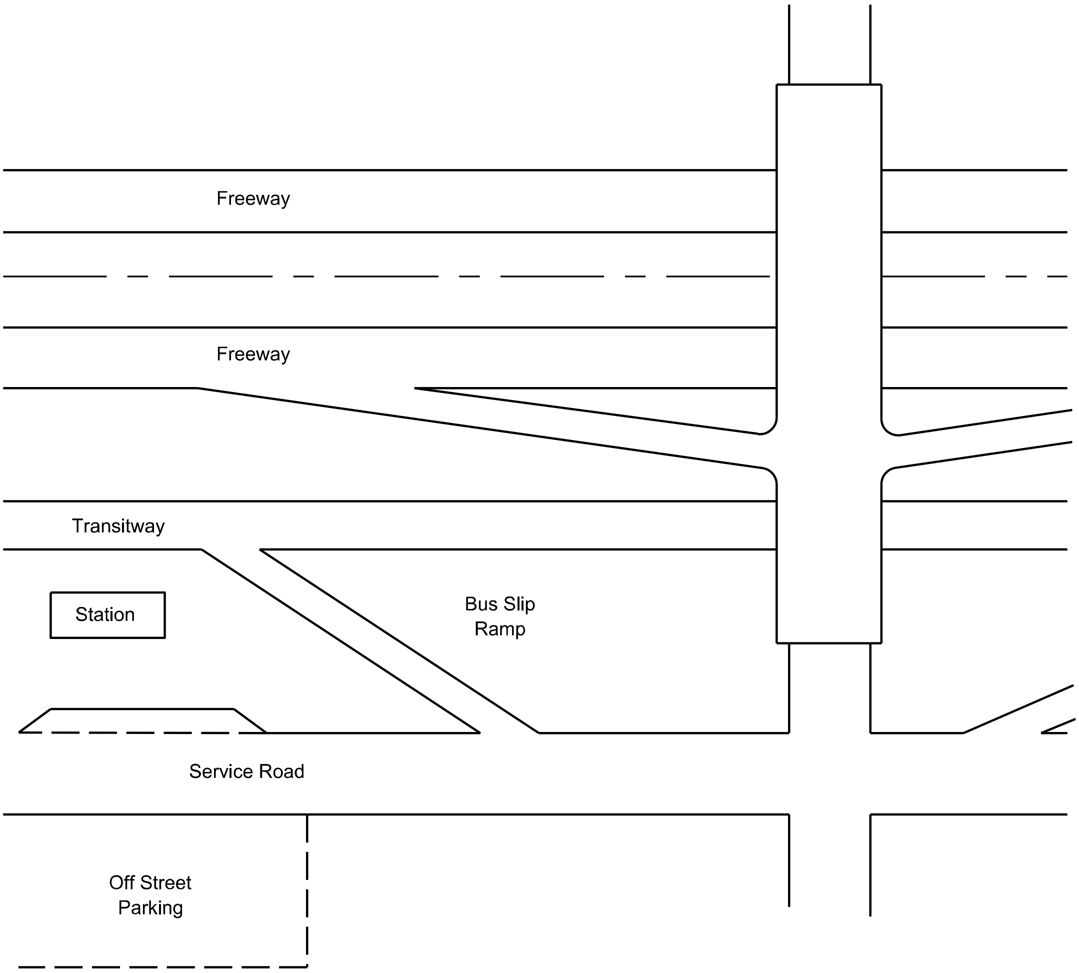
Combining mass transit or managed-lane facilities with freeways is a means for providing optimum transportation services in larger cities. This type of improvement can be accomplished by the joint use of right-of-way to include rail transit or separate roadway facilities for managed lanes. See Figure 7-31. The total right-of-way cost not only is less than those for two separate land strips, but the combination also preserves taxable property, reduces the displacement of businesses and persons, and lessens impact on neighbourhoods. In some cases, mass transit is incorporated into existing freeway systems.

Reverse-flow roadways, see Section 7.3.5 and Figure 7-32, in the median and reserved lanes work well for exclusive bus and high-occupancy vehicle use during rush hours. Bus roadways within the median essentially restrict operations to the line-haul or express type, because ramps that would permit collection and distribution from the median area are expensive or operationally undesirable. Furthermore, when freeways undergo major repair or reconstruction, it is often desirable to construct crossovers and temporarily shift all traffic onto one roadway. Where transit is located within the median, such temporary crossovers are not practical without disruption of transit operations.

7.7.4.2 Buses

Many metropolitan areas have nonstop freeway express buses that operate on the freeway system between outer urban pickup points near the freeway and destinations within the central business district or to other heavy traffic generators. The number of buses operating during peak hours, the spacing of bus stops, and the design of bus turnouts determine the efficiency of bus operation and its effect on roadway operations. Buses operating with short headways and frequent pickup and discharge points are likely to accumulate at stops and interfere with through traffic. On the other hand, express bus operation with few, if any, stops along the freeway provides superior transit service for outer urban areas and affects freeway operation the least.

Figure 7-31. Bus Transitway Located Between a Freeway and a Parallel Service Road



Source: (1)

Figure 7-32. Reverse Flow Lanes on a Separated Roadway

7.7.4.3 HOV Lanes

In addition to express service, other operational means should be considered to reduce the travel time of the public transportation user when demand warrants. An exclusive HOV roadway is an entire roadway facility reserved at all times solely for the use of buses and/or other HOVs. HOV facilities offer buses and HOVs a high level of service and decreases travel time for the users.

In general, HOV lanes are congestion dependent improvements (i.e. unless extreme congestion occurs regularly on the freeway mainlane, HOV lanes will not be successful in generating significant new carpooling and transit usage). Desirably, HOV facilities should be a part of a complete ridesharing program including park-and-ride lots and information services.

While an analysis of HOV demands is required, the following general conditions should exist or be projected before serious consideration is given to HOV lane alternatives:

- The HOV facility should be a part of an overall transportation plan.
- Intense, recurring congestion must exist on the freeway general purpose lanes.
- Peak period traffic per lane on the freeway should be approaching capacity.
- During the peak hour, average speeds on the freeway mainline during non-incident conditions should be less than 50 km/h over a distance of 8 km or more.
- Relative to the using the freeway general purpose lanes, the HOV facility should provide a time saving of at least 5 to 7 minutes.
- Of the peak-period trips on the freeway destined to major activity centres, at least 65% to 75% should be longer than 8 km in length.
- The resulting rideshare demand should sufficient to generate HOV vehicular volumes that are high enough to make the facility appear to be adequately utilized. The specific volume will vary depending upon the HOV facility. For bus-only facilities, the peak period transit volumes range from 100 to 150 buses per hour; for concurrent-flow lanes, peak-hour volumes should be in the range of 400 to 500 vehicles per hour; and, for separate facilities, suggested volumes should be 600 to 800 vehicles per hour in the HOV lane.

Within the freeway right-of-way, the following HOV facilities may be considered:

1. <u>Separate Roadways</u>. Separate roadways are usually located in the median of the freeway; see Figure 7-32. Single lane separated roadways are commonly one-way reversible and operate in the peak direction of travel. Multi-lane HOV facilities may be either two-way or one-way reversible. Reversible lanes are provided on freeways that have a high directional split (e.g. 60/40 or higher split) during the peak hour.
2. <u>Concurrent Flow Lanes</u>. Concurrent flow lanes are the preferred approach when two-way operation is desirable and space precludes the implementation of a separate roadway; see Figure 7-33. Concurrent flow lanes allow for opportunity for frequent access. Contiguous lanes are suitable for HOV operations where the HOV lane reverts back to mix flow lanes during non-peak periods. Lack of physical separation and ease of access for users and non-users increases the need for enforcement.

Figure 7-33. HOV Lane (Queen Elizabeth Way - Burlington to Toronto, Canada)

3. Contraflow Lanes. For contraflow facilities, one of the mixed flow lanes in the off-peak travel direction is dedicated to be used by HOVs travelling in the peak direction during peak periods. If the off-peak directional volume is relatively low during the peak period and, if mixed flow traffic can still be accommodated in the off-peak direction during the peak period on one lane less than what presently exist, then a contraflow lane may be feasible.

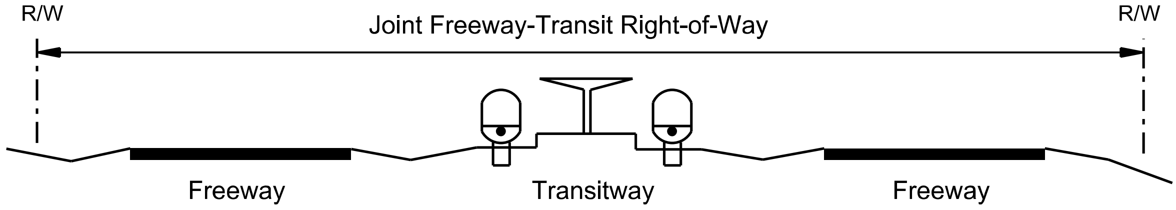
For additional guidance, HOV lanes and roadways are discussed in the AASHTO *Guide for High-Occupancy Vehicle (HOV) Facilities* (39). A discussion of the park-and-ride facilities that are often provided with HOV lanes is contained in the AASHTO *Guide for the Design of Park-and-Ride Facilities* (40).

7.7.4.4 Rail Transit

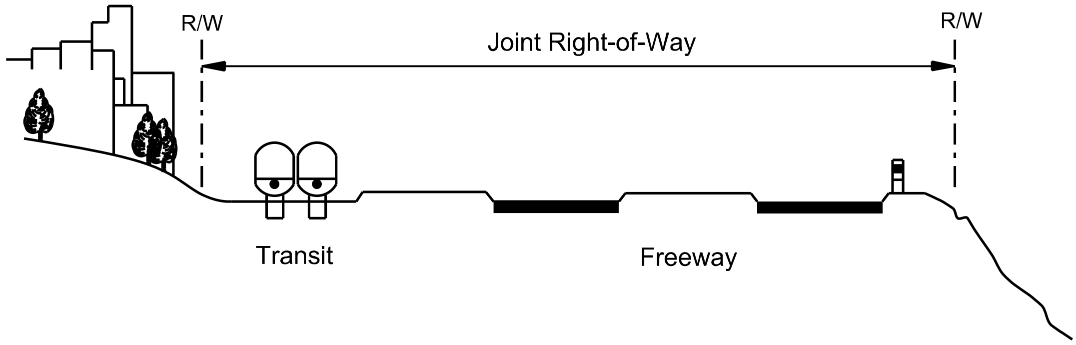
Several metropolitan areas have incorporated, or plan to incorporate, rail transit into freeway rights-of-way. Figure 7-34 illustrates various arrangements of the joint freeway-transit use of a right-of-way.

Location and design of rail transit facilities are joint undertakings involving several specialised fields of interest. The location and design of stations, terminals, and parking facilities should be considered from the standpoint of serving these facilities by urban streets and roads. Where rail is continuous to the freeway travelled way, the entire roadway design is affected.

Figure 7-34: Joint Freeway-Transit Right-of-Way



(a)



(b)

Source: (1)

8 RURAL ROADS

8.1 Overview

For the purposes of this Manual, rural roads are those roads outside of the urban areas defined by the maps contained in Plan Abu Dhabi 2030, Plan Al Ain 2030, and Plan Al Gharbia 2030. The design of rural roads covers a broad range of functional classifications from two-lane collectors and local roads to multilane arterials and expressways. Although there are rural freeways, they have distinctive design criteria and, therefore, are treated separately in Chapter 7 "Rural and Urban Freeways."

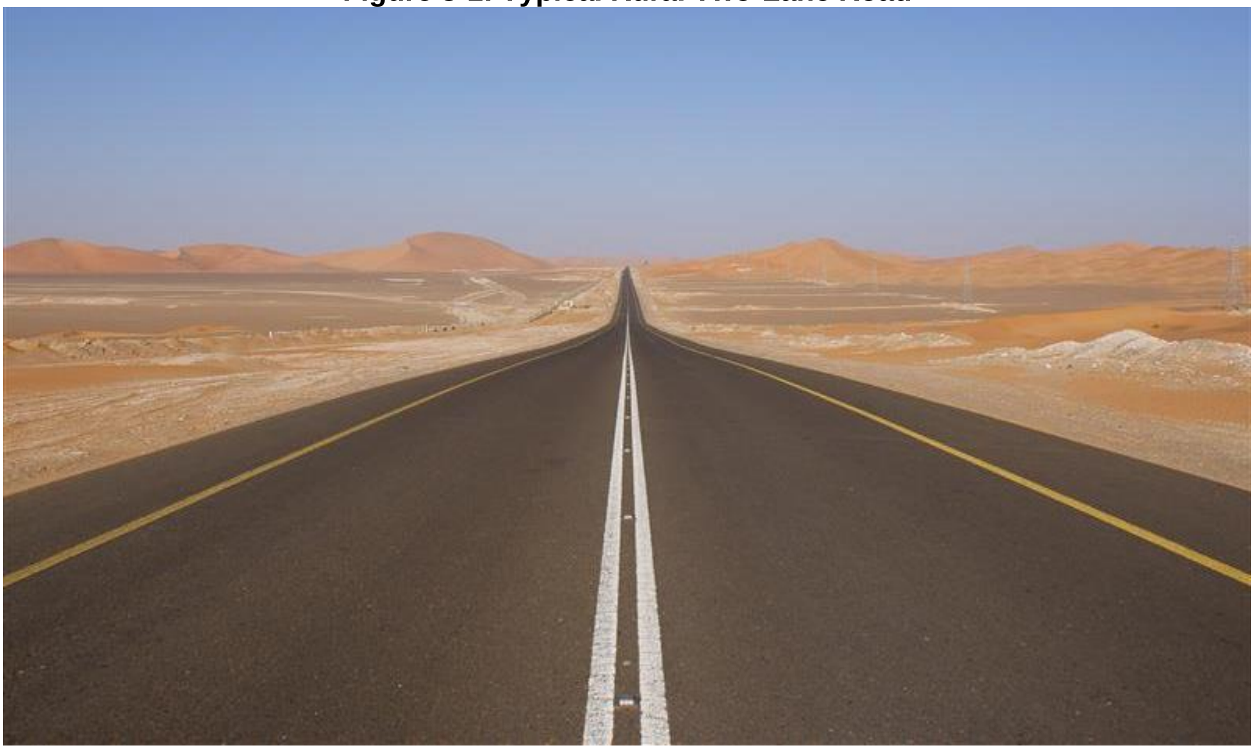
This chapter provides guidance on the application of geometric design criteria for rural two-lane roads and multilane roads separately because each has distinctive features. However, the designer should consider the design features from both types to provide for suitable transitions as the roadway changes between two-lane to multilane designs.

8.2 Two-Lane Roads

8.2.1 General Characteristics

Two-lane roads constitute an important part of the rural roadway system. They serve primarily to provide access to farms, residences, businesses, or other abutting properties. These roadways should be designed to accommodate the highest practical criteria compatible with traffic and topography. Figure 8-1 illustrates a typical rural two-lane roadway.

Figure 8-1: Typical Rural Two-Lane Road



The appropriate design geometrics for a two-lane road may be readily determined from the selected design speed, design traffic volumes, type of terrain, general character of the alignment, and composition of traffic. Table 8-1 and Table 8-2 provide the geometric design criteria for rural, two-lane collectors and local roads, respectively. The following sections provide additional guidance on the geometric design elements.

It may not be cost-effective to design rural roads that carry 400 vehicles per day or less using the same criteria applicable to higher volume roads or to make extensive traffic operational or safety improvements to such very low-volume roads. These roads are primarily used by motorists who travel them frequently and are familiar with their geometric design features. The unique characteristics of these roads are generally accepted and anticipated by the drivers using them. For these low-volume roads, the designer may want to design the road in accordance with criteria provided in the AASHTO *Guidelines for Geometric Design of Very Low Volume Local Roads (ADT ≤ 400)* (41)

8.2.2 General Design Considerations

8.2.2.1 Design Speed

The design speed for two-lane roads will vary depending on the functional classification of the road, terrain, and driver expectancy. Geometric design features should be consistent with a design speed appropriate for the conditions. Table 8-3 and Table 8-4 provide the design speeds for rural, two-lane collectors and local roads, respectively. The designer should strive for higher speeds than those shown where costs are not prohibitive.

8.2.2.2 Design Traffic Volume

Before an existing road is improved or a new rural road is constructed, the design traffic volume should be determined. The first step in determining the design traffic volume is to determine the current average daily traffic (ADT) volume for the roadway; in the case of new construction, the ADT can be estimated. The ADT volume should then be projected to the design year, usually 20 years after the anticipated construction completion. The design of two-lane, rural roads is typically based on ADT volumes alone because neither capacity nor intersection operations typically govern the overall operation. These roadways normally experience free-flow traffic under all conditions. By contrast, it is usually appropriate to design high-volume rural roads using an hourly volume for the design. The design hourly volume (DHV) should be the 30th highest hourly volume of the design year, which is typically about 15% of the ADT on rural roads.

Table 8-1: Geometric Design Criteria — Rural Collectors

Design Element				Manual Section	Design Criteria
Design Controls	Design Year			2.7	20 Years
	Design Speed			8.2.2	See Table 8-3
	Access Control			2.10	Direct Access
	Level of Service			2.7	Des: C Min: D
Cross Section Elements	Travelled Way	Lane Width		3.2	3.65 m
		Surface Type		3.2	Paved
	Shoulder Width	Total Width **(1)**		3.2	1.2 m
		Paved Width		3.2	Min: 0.5 m
		Surface Type		3.2	Paved/Aggregate
	Auxiliary Lanes	Lane Width		3.2	3.65 m
		Shoulder Width		3.2	Desirable: Full Shoulder Width Minimum: 1.2 m
	Cross Slope	Travel Lane		3.2	2% Typical
		Shoulder		3.2	2% Typical **(2)**
		Auxiliary Lanes		3.2	2% Typical
	Right-of-Way Width			3.6	Major Collector: 65 M Minor Collector: 57 m **(3)**
Roadway Slopes	Side Slopes	Cut Section **(4)**	Front Slope	3.4	1:6
			Ditch Width		2.4 m
			Back Slope		1:3
		Rock Cut		3.4	See Section 3.4
		Fill Section	Height: 0-1 m	3.4	1:6
			Height: 1-3 m		1:6 to Clear Zone; 1:4 to Toe
			Height: >3 m		1:6 to Clear Zone; 1:3 to Toe
	Verges			3.4	1.0 m
Bridges	Clear Roadway Width			8.2.4	See Table 8-6
	Vertical Clearance (Collector under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)			6.4	6.5 m
	Vertical Clearance (Collector over Railroad)			6.4	7.5 m

Alignment			Manual Section	**Design Speed**				
				50 km/h	**60 km/h**	**70 km/h**	**80 km/h**	**100 km/h**
	Stopping Sight Distance (Level)		4.3	65 m	85 m	105 m	130 m	185 m
	Decision Sight Distance		4.4	145 m	170 m	200 m	230 m	315 m
	Passing Sight Distance		4.5	160 m	180 m	210 m	245 m	320 m
	Minimum Radii	$e_{max} = 0.06$	5.2	79 m	123 m	184 m	252 m	438 m
		$e_{max} = 0.08$		73 m	113 m	168 m	229 m	394 m
	Vertical Curvature (K-values)	Crest	6.3	7	11	17	26	52
		Sag	6.3	13	18	23	30	45
	Maximum Grade	Level	6.2	7%	7%	7%	6%	5%
		Rolling	6.2	9%	8%	8%	7%	6%
		Mountainous	6.2	10%	10%	10%	9%	8%
	Minimum Grade		6.2	Desirable: 0.3% — 0.5% Minimum: 0% with proper drainage				

1. Shoulder Width. All shoulder widths should desirably be increased by 0.6 m when a barrier is present.
2. Shoulder Cross Slope. Where an aggregate shoulder is part of the shoulder width, slope the aggregate portion of the shoulder at 6%.
3. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths, plus the shoulder widths, plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus a service corridor.
4. Cut Slopes. Typical values shown in table apply to earth cuts; see Section 3.4 for rock cuts.

Table 8-2: Geometric Design Criteria — Rural Local Roads

	Design Element			Manual Section	Design Criteria				
Design Controls	Design Year			2.7	20 Years				
	Design Speed			8.2.2	See Table 8-4				
	Access Control			2.10	Direct Access				
	Level of Service			2.7	Des: C Min: D				
Cross Section Elements	Travelled Way	Lane Width		3.2	3.65 m **(1)**				
		Surface Type		3.2	Paved				
	Shoulder Width	Total Width **(2)**		3.2	Not Applicable				
	Cross Slope	Travel Lane		3.2	2% Typical				
		Shoulder		3.2	2% Typical **(3)**				
		Auxiliary Lanes		3.2	2% Typical				
	Right-of-Way Width			3.6	36 m **(4)**				
Roadway Slopes	Side Slopes	Cut Section **(5)**	Front Slope	3.4	1:6				
			Ditch Width		2.4 m				
			Back Slope		1:2				
		Rock Cut		3.4	See Section 3.4.				
		Fill Section	Height: 0-1 m	3.4	1:6				
			Height: 1-3 m		1:6 to Clear Zone; 1:3 to Toe				
			Height: >3 m		1:6 to Clear Zone; 1:3 to Toe				
	Verges			3.4	0.6 m				
Bridges	Clear Roadway Width			8.2.4	See Table 8-6				
	Vertical Clearance (Local Road under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)			6.4	6.5 m				
	Vertical Clearance (Local Road over Railroad)			6.4	7.5 m				
Alignment					**Design Speed**				
					40 km/h	**50 km/h**	**60 km/h**	**70 km/h**	**80 km/h**
	Stopping Sight Distance (Level)			4.3	50 m	65 m	85 m	105 m	130 m
	Decision Sight Distance			4.4	—	145 m	170 m	200 m	230 m
	Passing Sight Distance			4.5	140 m	160 m	180 m	210 m	245 m
	Minimum Radii		e_{max} = 0.06	5.2	43	79 m	123 m	184 m	252 m
			e_{max} = 0.08		41	73 m	113 m	168 m	229 m
	Vertical Curvature (K-values)		Crest	6.3	4	7	11	17	26
			Sag	6.3	9	13	18	23	30
	Maximum Grade		Level	6.2	7%	7%	6%	7%	6%
			Rolling	6.2	11%	10%	10%	8%	7%
			Mountainous	6.2	15%	14%	13%	10%	9%
	Minimum Grade			6.2	Desirable: 0.3% — 0.5% Minimum: 0% with proper drainage				

1. Travel Lane Widths. On low-volume roads (i.e. less than 400 vehicles/day), lane widths may be 3.0 m.
2. Shoulder Width. A 0.6m shy should desirably be inserted when a barrier is present.
3. Shoulder Cross Slope. Where an aggregate shoulder is part of the shoulder width, slope the aggregate portion of the shoulder at 6%.
4. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths, plus the shoulder widths, plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus a utility strip.
5. Cut Slopes. Typical values shown in table apply to earth cuts; see Section 3.4 for rock cuts.

Table 8-3: Design Speed — Rural Collectors

Type of Terrain	Design Speed (km/h) for Design Volume (vehicles/day)		
	0 to 400 (veh/day)	400 to 2000 (veh/day)	Over 2000 (veh/day)
Level	80	80	100
Rolling	60	70	80
Mountainous	50	60	70

Source: (1) *revised*

Table 8-4: Design Speed — Rural Local Roads

Type of Terrain	Design Speed (km/h) for Design Volume (vehicles/day)					
	Under 50	50 to 250	250 to 400	400 to 1500	1500 to 2000	2000 and over
Level	50	50	60	60	80	80
Rolling	40	50	50	60	60	70
Mountainous	30	30	30	50	50	60

Source: (1) *revised*

8.2.2.3 Capacity Analysis

Procedures for estimating the traffic operational performance for roadway designs are presented in the *Highway Capacity Manual 2010* (HCM) (17). The designer should consider the following when conducting a capacity analysis for rural, two-lane roadways:

1. Classes of Roadways. To address the wide range of functions served by two-lane roadways, the *HCM* segregates rural, two-lanes roads as followings:

   a. Class I. Class I roads are roads where motorists' speeds are expected to be relatively high. Two-lane roadways that are daily commuter links, major intercity roads, primary connectors of major traffic generators, or major segments in the national highway network are generally assigned to Class I.

   b. Class II. Class II roads are roads where motorists usually do not travel at fast speeds. Two-lane roadways that act as access routes to Class I roads, serve as scenic/recreational links, or pass through rugged terrain are designated to Class II.

   c. Class III. Class III roads are roads providing service to average developed areas. They may be portions of a Class I or Class II roadway that travel through small towns or developed recreational areas. On these facilities, local traffic often mixes with through traffic and the density of the unsignalised roadside access points is easily recognised as higher than in a true rural area.

2. Capacity. Under typical conditions, a two-lane roadway has a one direction capacity of 1100 passenger cars per hour (pc/h).

3. Level of Service (LOS). Designers should strive to provide the highest LOS practical and consistent with anticipated conditions. Table 8-1 and Table 8-2 provide the design level of service criteria for rural collectors and local roads, respectively.

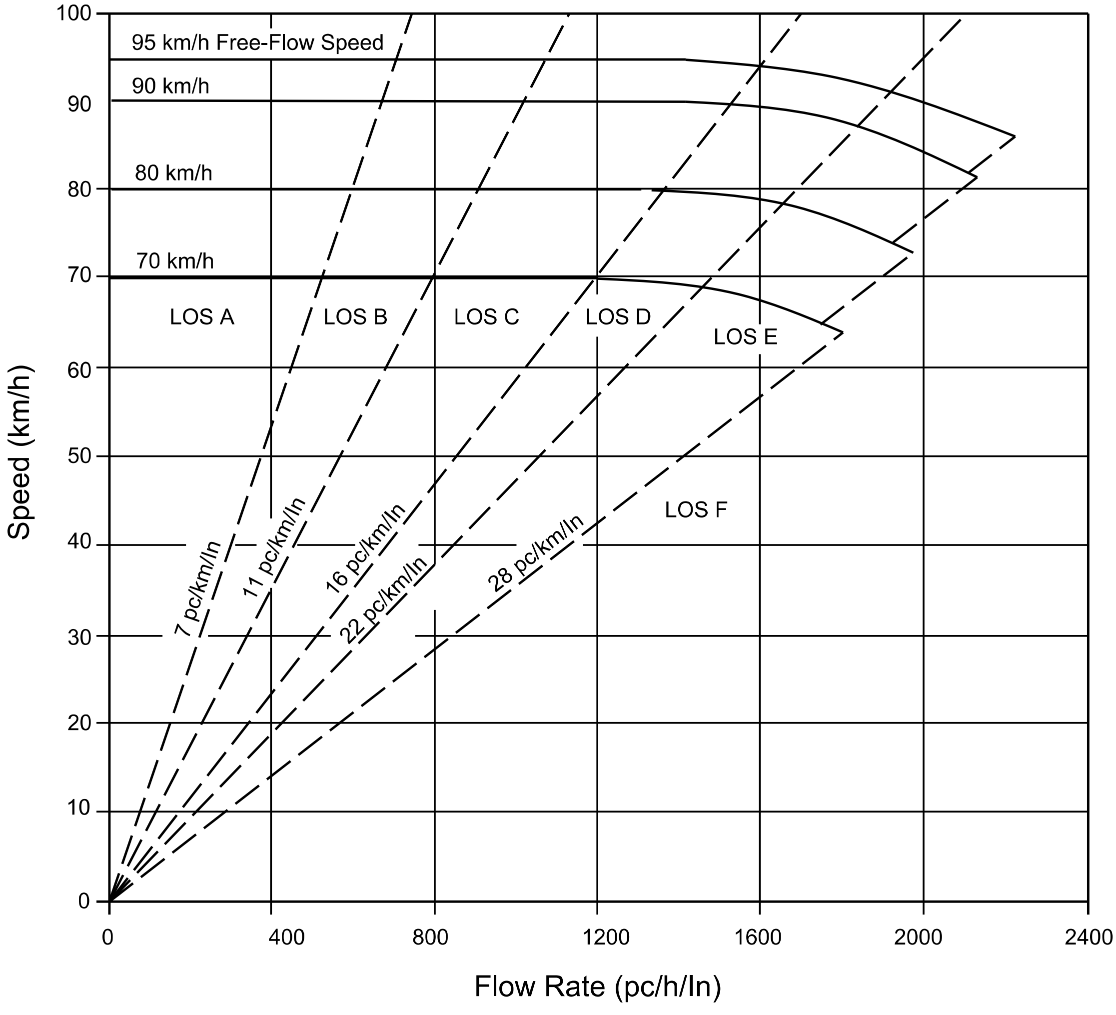
   For drivers on Class I two-lane roadways, speed and delay due to passing restrictions are both important; therefore, LOS is defined as both Average Travel Speed (ATS) and Percent Time-Spent-Following (PTSF). For drivers on Class II roadways, travel speed is not a significant issue; therefore, LOS is defined in terms of PTSF only. High travel speeds are not expected for Class III roadways. Passing restrictions are not a major concern because the length of Class III roadway segments is typically limited. Motorists would prefer to drive at or near the speed limit; therefore, Percent of Free-Flow Speed (PFFS) is used to define LOS. The LOS criteria for two-lane roadways are shown in Table 8-5 and Figure 8-2.

Table 8-5: LOS for Two-Lane Roadway

LOS	Class I Highways		Class II Highways PTSF (%)	Class III Highways PFFS (%)
	ATS (km/h)	PTSF (%)		
A	> 90	≤ 35	≤ 40	> 92
B	> 80 – 90	> 35 – 50	> 40 – 55	> 83 – 92
C	> 70 – 80	> 50 – 65	> 55 – 70	> 75 – 83
D	> 60 – 70	> 65 – 80	> 70 – 85	> 67 – 75
E	≤ 60	> 80	> 85	≤ 67

Source: (17)

Figure 8-2: LOS on Base Speed-Flow Curves



8.2.2.4 Sight Distance

Sight distance is directly related to and varies appreciably with design speed. Stopping sight distance should be provided throughout the length of the roadway. Passing and decision sight distances influence roadway operations and should be provided wherever practical.

Providing decision sight distance at locations where complex decisions are made greatly enhances the capability for drivers to safely accomplish manoeuvres. Example locations where complex decisions may be necessary include interchanges, high-volume intersections, transitions in roadway width, and transitions in the number of lanes. See Chapter 4 “Sight Distance” for a comprehensive discussion of stopping, passing, and decision sight distances.

8.2.2.5 Alignment

The designer should provide the most favourable alignment as practical for rural roads. Horizontal and vertical alignment should complement each other and should be considered in combination to achieve appropriate safety, capacity, and appearance for the type of improvement proposed. Topography, traffic volumes and composition, and right-of-way conditions are controlling features. Avoid abrupt changes in horizontal and vertical alignment.

Where curves are provided, a superelevation rate based on the design speed should be used. Maximum superelevation rates should not exceed 6% to 8% for rural collectors and local roads. Superelevation runoff denotes the length of roadway needed to accomplish the change in cross slope from a section with adverse crown removed to a fully superelevated section and vice versa. Adjustments in design runoff lengths may be needed for smooth riding, drainage, and appearance. Chapter 5 “Horizontal Alignment” provides a detailed discussion of superelevation and tables of appropriate superelevation rates and runoff lengths for various design speeds.

Vertical curves should meet the sight distance criteria for the design speed. In addition, it is desirable to provide frequent opportunities for passing; see Section 8.2.5.

For further information on alignment designs, see Chapter 5 “Horizontal Alignment” and Chapter 6 “Vertical Alignment.”

8.2.2.6 Grades

The length and steepness of grades directly affect the operational characteristics of a road. Table 8-1 and Table 8-2 provide the maximum grades for rural collectors and local roads, respectively.

8.2.3 Cross Section Elements

Figure 3-3 shows a typical cross section for rural roadways.

8.2.3.1 Road and Shoulder Widths

Travel lane widths on collectors and high-volume local roads should be 3.65 m. On low-volume, rural local roads, travel lane widths as narrow as 3.0 m may be used.

Refer to Table 8-1 and 8-2 for shoulder widths.At a minimum, 0.5 m of the shoulder width should be paved to provide for pavement support, space for wide vehicles, and collision avoidance. Where cyclists are to be accommodated on the shoulder, the designer should provide a minimum paved width of 1.5 m. The shoulder should be constructed to a uniform width for relatively long stretches of the roadway.

For additional information concerning travel lanes and shoulders widths, see Chapter 3 “Cross Section Elements.”

8.2.3.2 Cross Slopes

Roadway cross slopes provide for proper drainage. Normally, cross slopes are 2% for paved two-lane roadways, with the crown at the roadway centreline. However, if the ultimate roadway is planned to be a dual carriageway, the designer should provide a uniform 2% cross slope across the travelled way and shoulders sloping away from the future median.

8.2.3.3 Roadside

The Abu Dhabi *Roadside Design Guide* (28) provides the minimum clear zone distances based on the design speed, traffic volumes, and side slopes. The clear zone should be clear of all unyielding objects (e.g. trees, non-breakaway sign supports, utility poles, luminaries) and other fixed objects.

Roadside slopes should be as flat as practical, taking into consideration other design elements. Flat front slopes reduce potential crash severities, aid in the establishment of plant growth, reduce the effects of blowing sand, and reduce maintenance operations. Table 8-1 and Table 8-2 provide the side slope criteria for rural collectors and local roads, respectively. In addition, the designer should check the geotechnical report to determine the appropriate side slopes based on the local site conditions.

8.2.3.4 Right-of-Way

The right-of-way is typically configured to accommodate all proposed cross-section elements throughout the project, including the ultimate proposed cross section (including a utility strip). This usually precludes a uniform right-of-way width because there are typically many situations where additional width is advantageous. Such situations may occur where the side slopes extend beyond the normal right-of-way, for clear areas at the bottom of traversable slopes, for wider clear areas on the outside of curves, where greater sight distance is desirable, at intersections and junctions with other roads, at railroad-roadway grade crossings, for environmental considerations, and for maintenance access.

Where additional lanes are proposed to be completed in the future, the initial right-of-way width should be adequate to provide for the wider roadway section including widths for utility services. Design the initial two lanes off centre within the right-of-way so that future construction will cause less interference with traffic and the investment in initial grading and surfacing can be salvaged.

8.2.4 Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the Abu Dhabi *Road Structures Design Manual* (29).

Desirably, the full width of the roadway including shoulders, footpath and cycle track on the approach roadways shall continue on the bridges.

Road overpasses, sign structures, and pedestrian bridges will have a 6.5-m clearance over the entire roadway width including the usable shoulder width. Overpasses over railroads require a 7.5-m clearance. The designer should consider providing an additional clearance to allow for future resurfacing.

Table 8-6: Minimum Clear Roadway Width on Bridge — Rural Collectors and Local Roads

Design Volume (vehicles/day)	Minimum Clear Roadway Width for Bridges(1)
400 and under	Travelled Way Width + 1.2 m (each side)
400 to 2000	Travelled Way Width + 1.8 m (each side)
Over 2000	Approach Roadway Width(2)

Notes:

1. *Where the approach roadway width (travelled way plus shoulders) is paved, that surface width should be carried across the structure.*

2. *Sidewalk may be required for maintenance and pedestrians.*

Source: (1) *revised*

8.2.5 Provision for Passing

8.2.5.1 Operations

In designing two-lane, two-way roads, it is desirable to provide alignment and profile sections suitable for passing at frequent intervals. The criteria in Table 8-1 and Table 8-2 provide the minimum passing sight distances. Design of the horizontal and vertical alignments should provide adequate passing sight distance over as large a proportion of the roadway length as practical. Short passing sections have been found to contribute little to improving the traffic operational efficiency of a two-lane roadway. Passing sections less than the minimums shown in Table 8-7 should be excluded from consideration when determining the level of service for the two-lane roadway.

Table 8-7: Minimum Passing Zone Length for Traffic Operational Analyses

Design Speed Limit (km/h)	Minimum Passing Zone Lengths (m)
40	140
50	180
60	210
≥ 70	240

Source: (1) *revised*

Restrictive cases may exist where passing sight distances are economically difficult to justify (e.g. mountainous terrain). Even in these instances, passing opportunities should be provided with at least the frequency needed to attain the desired level of service. Where achievement of sufficient passing sight distance is not practical, auxiliary lanes (e.g. truck climbing lanes, passing lanes) should be considered as a means to obtain the desired level of service.

In summary, the designer should consider the following to provide passing opportunities on two-lane roadways:

1. Design of the horizontal and vertical alignments should provide as large a proportion of the road length as practical with adequate passing sight distance.
2. For design volumes approaching capacity, consider the effect passing opportunities have on increasing capacity.
3. Consider providing climbing lanes. For further information for climbing lane warrants, see Chapter 6 “Vertical Alignment.”
4. Where the extent and frequency of passing opportunities made available by application of Items 1 and 3 are insufficient, the designer should consider providing an added lane to improve traffic operations.

8.2.5.2 Passing Lanes

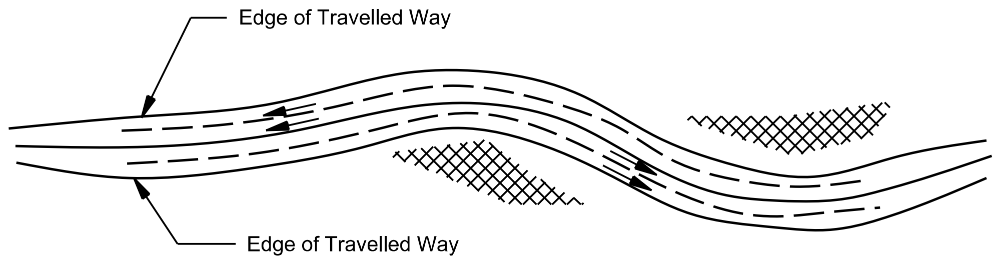
Passing lanes, other than truck-climbing lanes, may be warranted on two-lane facilities where passing opportunities are not adequate as determined based on an engineering study or from an operational experience.

An added lane can be provided in one or both directions of travel to improve traffic operations in sections of lower capacity. Passing-lane sections may be either three or four lanes in width. Occasional passing lanes may improve overall traffic operations on two-lane roads by reducing delays caused by inadequate passing opportunities over significant lengths of the roadway, see Figure 8-3.

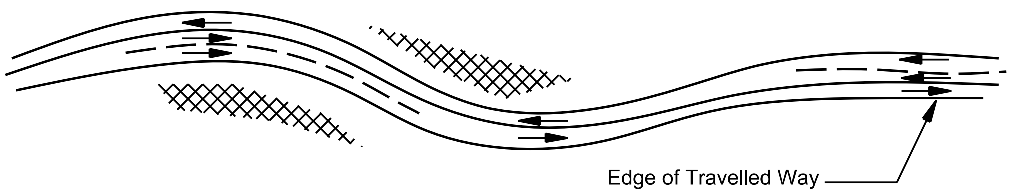
The designer should consider the following when designing passing lanes:

1. Location. The location of the added lanes should appear logical to the driver. Consider providing passing lanes where platooning begins to significantly affect roadway operations.
2. Sight Distance. The location of a passing lane should provide adequate sight distance at both the lane addition and lane drop tapers. Provide a minimum sight distance of 300 m on the approach to the beginning and ending tapers.
3. Width. The added travel lane width is 3.65 m. Desirably, the shoulder width through the passing section will be same as the approach shoulder width; however, a minimum 1.2 m wide shoulder width may be provided in restricted situations.

Figure 8-3: Passing Lanes on a Two-Way, Single Carriageway



a) Four-Lane Passing Section on Two-Lane Roadway



b) Three-Lane Passing Section on Two-Lane Roadway

4. Length. To allow sufficient opportunities for passing, the minimum length of the passing lane should be 500 m, excluding the taper. Lengths from 800 m to 1500 m typically provide the best operational benefits. Passing sections 3000 m or longer may cause drivers to lose their awareness that they are travelling on a two-lane roadway.

5. Median. Four-lane sections introduced solely to improve passing opportunities need not be divided because there is no separation of opposing traffic on the two-lane portions of the road. However, a median should be considered where the traffic volume is 500 vehicles/hour or more.

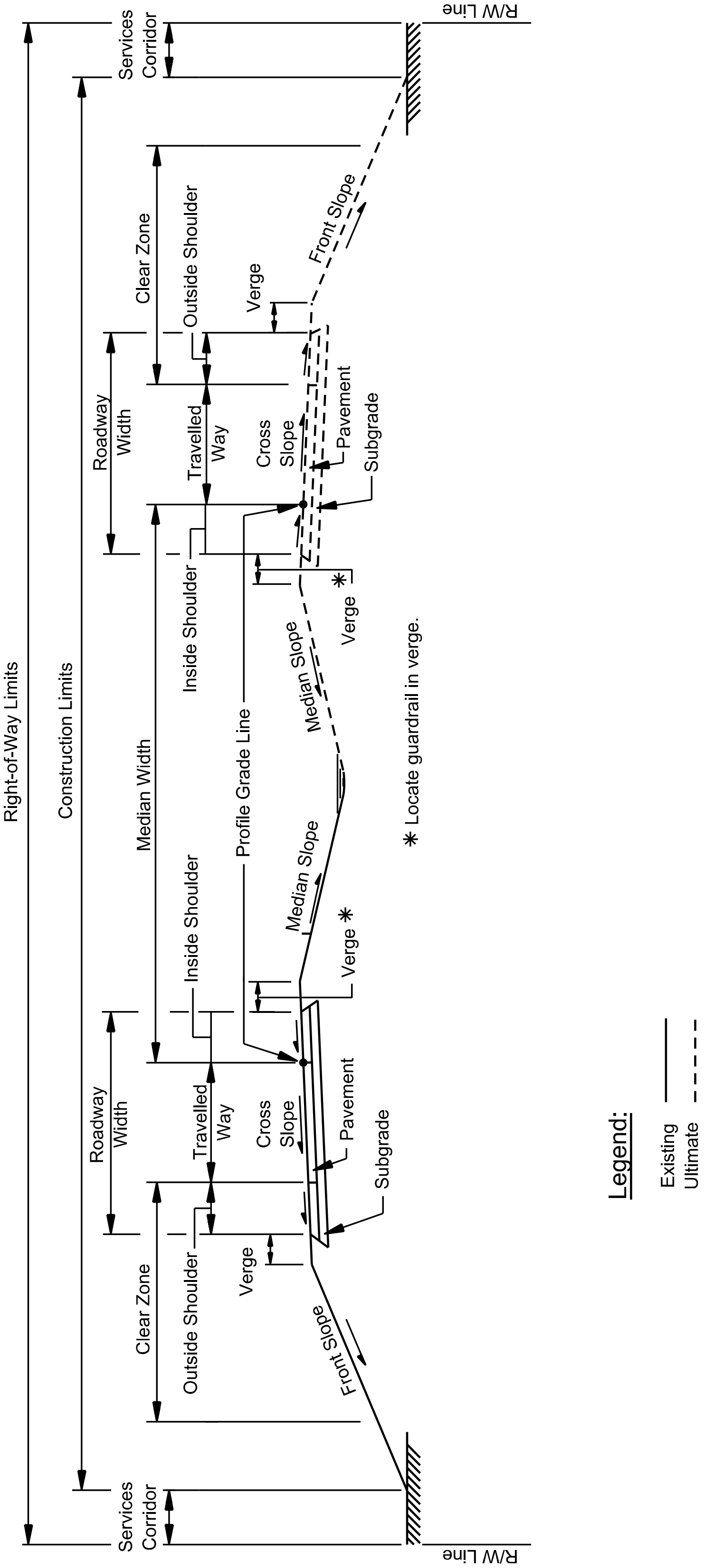
8.2.6 Ultimate Development for a Dual Carriageway

Although many two-lane roads will adequately serve future traffic demands, there are numerous instances, particularly near urban areas, where two-lane roads will ultimately need to be expanded to handle the expected traffic volumes.

Where it is anticipated that the DHV for the design year will exceed the service volume of the two-lane roadway for its desired capacity, the initial improvement should be consistent with the planned ultimate development of a 2+2 or 3+3 dual carriageway and should include acquisition of the needed right-of-way. See the *Highway Capacity Manual 2010* (17) for traffic operational analysis procedures to determine whether a two-lane roadway can provide the desired level of service or whether a four-lane roadway should be considered. The eventual need for additional lanes should be considered during the design of a two-lane road.

If the ultimate development is a dual carriageway, the initial two-lane roadway should be constructed so that it can eventually become one of the two carriageways; see Figure 8-4.

Figure 8-4: Single Carriageway Cross Section with Ultimate Development to Dual Carriageway



8.2.7 Turnouts

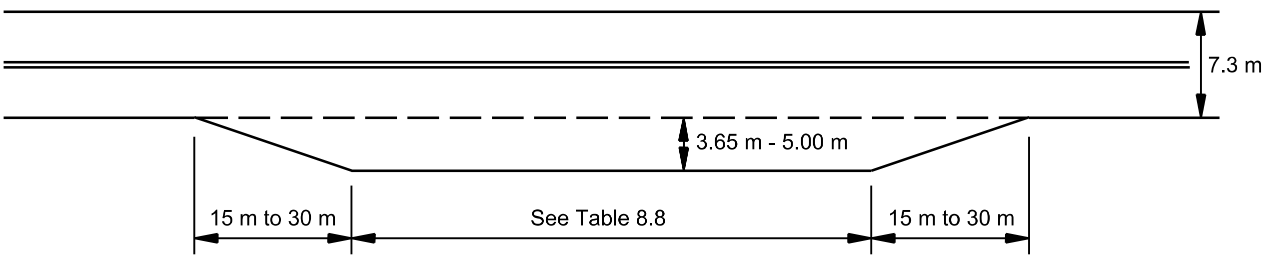
A turnout is a widened, unobstructed shoulder area that allows slow-moving vehicles to remove themselves from the through travel lane to allow following vehicles to pass. Where there are only a very small number of vehicles, the slow-moving vehicle can perform this manoeuvre without it being necessary to stop in the turnout area. However, if the number of following vehicles becomes too large, the driver of the slow-moving vehicle may need to stop in the turnout and allow the following vehicles to pass. Turnouts are most frequently used on lower volume roads where long platoons are rare and in challenging terrain with steep grades (e.g. where construction of an additional lane may not be cost-effective). These conditions are typically found where more than 10% of the vehicle traffic is large trucks.

The turnout lengths, including tapers, are shown in Table 8-8. Turnouts shorter than 60 m are not desirable even for very low approach speeds. Do not provide turnout lengths longer than 185 m for high-speed roads to avoid their use as a passing lane. The turnout lengths are based on the assumption that slow-moving vehicles enter the turnout at a speed 10 km/h slower than the mean speed of the through traffic. This length allows the entering vehicle to coast to the midpoint of the turnout without braking, and then, if necessary, to brake to a stop using a deceleration rate not exceeding 3 m/s^2. The lengths for turnouts include entry and exit tapers. Typical entry and exit tapers lengths range from 15 m to 30 m. Figure 8-5 illustrates a turnout design.

Table 8-8: Turnout Length

Approach Speed (km/h)	Minimum Length (m)
30	60
40	60
50	65
60	85
70	105
80	135
90	170
100	185

Figure 8-5: Turnout Design



The minimum width of the turnout is 3.65 m with widths of 5.0 m considered desirable. Do not use turnout widths greater than 5.0 m. Do not locate a turnout on or adjacent to a horizontal or vertical curve that limits sight distance in either direction.

Proper signing and pavement markings are also necessary to both maximise turnout usage and reduce crashes. An edge line marking on the right side of the turnout is desirable to guide drivers, especially in wider turnouts.

8.3 Rural Multilane Roads

Rural multilane roads (dual carriageways) are classified as arterials, expressways, or freeways. Because of their unique characteristics, freeways are discussed separately in Chapter 7 “Rural and Urban Freeways.” The appropriate design geometrics for arterials and expressways are determined from the selected design speed and the design traffic volumes, with consideration of the type of terrain, general character of the alignment and composition of traffic.

Rural expressways are generally multi-lane, divided roadways with both grade separated and at-grade intersections that carry large traffic volumes at high speed under close to free flow conditions. They connect and sometimes bypass cities and larger towns and serve industrial, recreational, and international traffic. To maintain the flow and safety of through traffic, direct property access is normally eliminated. Important crossroads may require grade separated interchanges. Figure 8-6 illustrates an expressway with an at-grade roundabout and grade separated interchanges.

Rural arterials are multi-lane roadways with at-grade intersections that carry large traffic volumes at high speeds. In conjunction with expressways, they connect major economic regions and centres (e.g. cities and towns, industrial concentrations).

Figure 8-6: Expressway with Grade Separations

Table 8-9 and Table 8-10 provide the geometric design criteria for rural expressways and arterials, respectively. The following sections provide additional guidance on the geometric design elements for rural dual carriageways.

Table 8-9: Geometric Design Criteria — Rural Expressways

Design Element				Manual Section	Design Criteria		
Design Controls	Design Year			2.7	20 Years		
	Design Speed			8.3.1	Des.: 120 km/h Min.:100 km/h **(1)**		
	Access Control			2.10	Limited Access		
	Level of Service			2.7	Des: C Min: D		
Cross Section Elements	Travelled Way	Lane Width		3.2	3.65 m		
		Surface Type		3.2	Paved		
	Shoulder Width **(2a)**	Right	Total Width	3.2	3.0 m **(2b)**		
			Paved Width	3.2	3.0		
			Surface Type	3.2	Paved		
		Left	Total Width	3.2	1.2 m		
			Paved Width	3.2	1.2 m		
			Surface Type	3.2	Paved		
	Auxiliary Lanes	Lane Width		3.2	3.65 m		
		Shoulder Width		3.2	Desirable: Full Shoulder Width Minimum: 1.8 m		
	Cross Slope	Travel Lane		3.2	2% Typical		
		Shoulder		3.2	2% Typical		
		Auxiliary Lanes		3.2	2% Typical		
	Median Width **(3)**	Depressed		3.3	Recommend: 10 m Desirable: 18 m		
		Flush (Concrete Barrier)		3.3	8.05 m		
	Right-of-Way Width			3.6	112 m **(4)**		
Roadway Slopes	Side Slopes	Cut Section **(5)**	Front Slope	3.4	1:6		
			Ditch Width	3.4	2.4 m		
			Back Slope	3.4	1:4		
		Rock Cut		3.4	See Section 3.4.		
		Fill Section	Height: 0-1 m	3.4	1:6		
			Height: 1-3 m	3.4	1:6 to Clear Zone; 1:4 to Toe		
			Height: >3 m	3.4	1:6 to Clear Zone; 1:3 to Toe		
	Verges			3.4	1.0 m		
Bridges	Clear Roadway Width			8.3.3	Full Approach Roadway Width		
	Vertical Clearance (Expressway Under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)			6.4	6.5 m		
	Vertical Clearance (Expressway over Railroad)			6.4	7.5 m		
Alignment					**Design Speed**		
					100 km/h	**110 km/h**	**120 km/h**
	Stopping Sight Distance (Level)			4.3	185 m	220 m	250 m
	Decision Sight Distance			4.4	315 m	330 m	360 m
	Minimum Radii		(e_{max} = 0.04)	5.2	492 m	635 m	872 m
			(e_{max} = 0.06)	5.2	438 m	561 m	756 m
	Vertical Curvature (K-values)		Crest	6.3	52	74	95
			Sag	6.3	45	55	63
	Maximum Grade (**6**)		Level	6.2	3%	3%	3%
			Rolling	6.2	4%	4%	4%
			Mountainous	6.2	6%	5%	5%
	Minimum Grade			6.2	Desirable: 0.3% — 0.5% Minimum: 0% with proper drainage		

1. Design Speed. Design speeds shown are for level terrain. A minimum design speed of 90 km/h may be used in rolling terrain. A minimum design speed of 80 km/h may be used in mountainous terrain.
2. Shoulder Width. The following will apply:
   a. Barriers. All shoulder widths should desirably be increased by 0.6 m when a barrier is present.
3. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e. planned addition of travel lanes), and field conditions.
4. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths (typically 3+3), plus the outside shoulder widths, plus the median width, plus the necessary width for fill and cut slopes and clear zones. The ROW width will accommodate the anticipated ultimate development of the facility plus utility services corridors.
5. Cut Slopes. Typical values in table apply to earth cuts; see Section 3.4 for rock cuts.
6. Maximum Grade. With wide medians where two roadways are on independent alignments, downgrades may be 1% steeper. Expressway grades in mountainous terrain may be 1% steeper.

Table 8-10: Geometric Design Criteria — Rural Arterials

	Design Element			Manual Section	Design Criteria
Design Controls	Design Year			2.7	20 Years
	Design Speed			8.3.1	Des.: 110 km/h Min.:90 km/h **(1)**
	Access Control			2.10	Limited Access
	Level of Service			2.7	Des: C Min: D
Cross Section Elements	Travelled Way	Lane Width		3.2	3.65 m
		Surface Type		3.2	Paved
	Shoulder Width **(2a)**	Right	Total Width**)**	3.2	3.0 m **(2b)**
			Paved Width	3.2	3.0
			Surface Type	3.2	Paved
		Left	Total Width**)**	3.2	1.2 m
			Paved Width	3.2	1.2 m
			Surface Type	3.2	Paved
	Auxiliary Lanes	Lane Width		3.2	3.65 m
		Shoulder Width		3.2	Desirable: Full Shoulder Width Minimum: 1.8 m
	Cross Slope	Travel Lane		3.2	2% Typical
		Shoulder		3.2	2% Typical
		Auxiliary Lanes		3.2	2% Typical
	Median Width **(3)**	Depressed		3.3	Recommend: 10 m Desirable: 18 m
		Flush (Concrete Barrier)		3.3	8.05 m
	Right-of-Way Width			3.6	Major Arterials: 82 m Minor Arterials: 72 m **(4)**
Roadway Slopes	Side Slopes	Cut Section **(5)**	Front Slope	3.4	1:6
			Ditch Width		2.4 m
			Back Slope		1:4
		Rock Cut		3.4	See Section 3.4.
		Fill Section	Height: 0-1 m	3.4	1:6
			Height: 1-3 m		1:6 to Clear Zone; 1:4 to Toe
			Height: >3 m		1:6 to Clear Zone; 1:3 to Toe
	Verges			3.4	1.0 m
Bridges	Clear Roadway Width			8.3.3	Full Approach Roadway Width
	Vertical Clearance (Expressway Under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)			6.4	6.5 m
	Vertical Clearance (Expressway over Railroad)			6.4	7.5 m

	Design Element		Manual Section	Design Speed		
				90 km/h	**100 km/h**	**110 km/h**
Alignment	Stopping Sight Distance (Level)		4.3	160 m	185 m	220 m
	Decision Sight Distance		4.4	270 m	315 m	330 m
	Minimum Radii	e_{max} = 0.04	5.2	375 m	492 m	635 m
		e_{max} = 0.06		336 m	438 m	561 m
	Vertical Curvature (K-values)	Crest	6.3	39	52	74
		Sag	6.3	38	45	55
	Maximum Grade (**6**)	Level	6.2	4%	3%	3%
		Rolling	6.2	5%	4%	4%
		Mountainous	6.2	6%	5%	5%
	Minimum Grade		6.2	Desirable: 0.3% — 0.5% Minimum: 0% with proper drainage		

1. Design Speed. Design speeds shown are for level terrain. A minimum design speed of 80 km/h may be used in rolling terrain. A minimum design speed of 60 km/h may be used in mountainous terrain.
2. Shoulder Width. The following will apply:
   a. Barriers. All shoulder widths should desirably be increased by 0.6 m when a barrier is present.
3. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e. planned addition of travel lanes), and field conditions.
4. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths, plus the outside shoulder widths, plus the median width, plus the necessary width for fill and cut slopes and clear zones. The ROW width will accommodate the anticipated ultimate development of the facility plus utility services corridors.
5. Cut Slopes. Typical values in table apply to earth cuts; see Section 3.4 for rock cuts.
6. Maximum Grade. With wide medians where two roadways are on independent alignments, downgrades may be 1% steeper. Grades in mountainous terrain may be 1% steeper.

8.3.1 General Design Considerations

8.3.1.1 Design Speed

Rural arterials and expressways should be designed for speeds of 60 km/h to 120 km/h depending on terrain and driver expectancy. Design speeds of 90 km/h to 120 km/h are normally used in level terrain; design speeds of 80 km/h to 100 km/h may be used in rolling terrain; and design speeds of 60 km/h to 80 km/h may be used in mountainous terrain.

8.3.1.2 Design Traffic Volumes

Before an existing road is improved or a new rural road is constructed, the designer needs to determine the design traffic volume. The first step in determining the design traffic volume is to determine the current average daily traffic (ADT) volume for the roadway; in the case of new construction, the ADT can be estimated. The ADT volume should then be projected to the design year, usually 20 years after the anticipated construction completion. It is usually appropriate to design rural multilane roads using an hourly volume for the design. The design hourly volume (DHV) should generally be the 30th highest hourly volume of the year, which is typically on rural roads about 15% of the ADT.

8.3.1.3 Alignment

The designer should provide the most favourable alignment as practical for rural roads. Horizontal and vertical alignment should complement each other and be considered in combination to achieve appropriate safety, capacity, and appearance for the type of improvement proposed. Topography, traffic volumes and composition, and right-of-way conditions are controlling features. Avoid abrupt changes in horizontal and vertical alignment.

Where horizontal curves are provided, use a superelevation rate based on the design speed. The maximum superelevation rate used to determine the design superelevation rate should be either 4% or 6% for expressways and 6% to 8% for other rural arterials. Chapter 5 “Horizontal Alignment” provides detailed discussions of superelevation, tables of appropriate superelevation rates and runoff lengths, and other horizontal alignment requirements.

Vertical curves should meet the sight distance criteria for the selected design speed. See Chapter 6 “Vertical Alignment.”

8.3.1.4 Level of Service

Procedures for estimating the traffic operational performance for roadway designs are presented in the *Highway Capacity Manual 2010* (17). The designer should consider the following when conducting a capacity analysis for multilane roadways:

1. <u>Base Conditions</u>. Uninterrupted flow on multilane roadways is in most ways is similar to that on basic freeways. However, lower design speeds, narrower cross sections, and at-grade intersections all combine to lower speeds and capacity on multilane roadways.

2. <u>Capacity</u>. Under ideal conditions, the capacity of an expressway and multilane arterial is 2000 passenger cars per hour per lane (pc/h/ln).

3. Level of Service (LOS). Designers should strive to provide the highest LOS practical and consistent with anticipated conditions. Table 8-9 and Table 8-10 provide the design LOS criteria for expressways and arterials, respectively. LOS for multilane highways is based on density, which is a measure of the proximity of vehicles to each other in the traffic stream.

   For LOS A through D, the criteria are the same as for basic freeway segments. The boundary between LOS D and E represents capacity and varies based on the free-flow speed (FFS). LOS F is where the demand flow rate exceeds capacity. The LOS criteria for multilane roadways are shown in Table 8-11.

Table 8-11: LOS for Multilane Highways

LOS	Free Flow Speed (km/h)	Density (pc/km/ln)
A	All	> 7*
B	All	> 7-11
C	All	> 11 – 16
D	All	> 16 – 22
E	72	> 22 – 28
	96	> 22 - 25
F	72 (V/C > 1.00)	> 28
	96 (V/C > 1.00)	> 25

Note: 1 Mile = 1.6 Km

Source: (17)

8.3.1.5 Grades

The length and steepness of grades directly affect the operational characteristics of an arterial. Table 8-9 and Table 8-10 present the criteria for maximum grades for rural expressways and arterials, respectively. When vertical curves for stopping sight distance are considered, there are seldom advantages to using the maximum grade values except where grades are long.

8.3.2 Cross Sectional Elements

8.3.2.1 Number of Lanes

The number of lanes on an arterial roadway should be determined based on consideration of traffic volumes, level of service, and capacity conditions. The typical number of lanes for rural expressways and arterials is three lanes (3+3) in each direction; however, two lanes (2+2) in each direction may be used.

8.3.2.2 Road and Shoulder Widths

Travel lane widths on arterials and expressways are 3.65 m wide. For truck routes, lane widths should be 3.75 m wide.

Table 8-9 and Table 8-10 provide the criteria for shoulder widths for rural expressway and rural arterials, respectively. The width of the usable outside shoulder should be at least 3.0 m. Paving the usable width of shoulder is desirable. Where cyclists are to be accommodated on the shoulder, the designer should provide a minimum paved width of 1.5 m. The cyclists are not allowed to use freeways and expressways.

Provide a paved inside shoulder width of 1.2 m. Where there are three or more lanes in each direction, the inside shoulder will be the same width as the outside shoulder width.

8.3.2.3 Cross Slopes

Travelled way cross slopes provide for proper drainage. Provide a cross slope of 2% uniformly sloped across the travelled way sloping away from the median. For cross sections with depressed medians, slope the inside (median) shoulder at 2% towards the median. For cross sections with flushed or raised medians, slope the inside shoulder at 2% in the same direction as the travelled way.

8.3.2.4 Medians

On roads without at-grade intersections, the median may be as narrow as 1.2 m under very constrained conditions, but wider medians should be provided wherever practical. A wide median allows the use of independent profiles. In addition, provision of a wide median may reduce the frequency of head-on crashes and reduce headlight glare from vehicles in the opposing direction of travel.

Although medians as narrow as 1.2 m may be used under very restricted conditions, medians 3.6 m to 9.0 m wide provide a protected storage area for left-turning vehicles at intersections. Where left turns are common, do not use a median width of 1.2 m to 3.6 m. These median widths do not provide sufficient space for turning vehicles and may encourage other motorists to encroach into the adjacent lane to avoid a turning vehicle that is only partially in the median.

In many cases, the median width at rural unsignalised intersections is a function of the design vehicle selected for the turning and crossing manoeuvres. Where a median width of 7.5 m or more is provided, a passenger car making a turning or crossing manoeuvre will have space to stop in the median area without encroaching on the through lanes. Medians less than 7.5 m wide should be avoided at rural intersections because drivers may be tempted to stop in the median with part of their vehicles unprotected from through traffic. The SU-30 design vehicle is often the largest vehicle to use the median crossing with any significant frequency. The selection of a SU-30 as the design vehicle results in a median width of 15 m. Larger design vehicles may be used at intersections where significant turning or crossing trucks are present. Median widths of at least 25 m or more may be necessary to accommodate large tractor-trailer trucks without encroaching on the through lanes of a major road.

Median widths over 18 m are undesirable at intersections that are signalised or may need signalisation in the foreseeable future. The efficiency of signal operations decreases as the median width increases, because drivers need more time to traverse the median. Special detectors may be needed to avoid trapping drivers in the median at the end of the green phase for traffic movements across the median. Furthermore, if the median is too wide separate, traffic signals are required on each roadway of the divided road. Delays to motorists will increase substantially. The designer will need to consider vehicle queues in the median between the two traffic signals.

8.3.2.5 Roadside

The Abu Dhabi *Roadside Design Guide* (28) provides the minimum clear zone distances based on the design speed, traffic volumes, and side slopes. The clear zone should be clear of all unyielding objects (e.g. trees, non-breakaway sign supports, utility poles, luminaries) and other fixed objects.

Roadside slopes should be as flat as practical, taking into consideration other design elements. Flat front slopes reduce potential crash severities, aid in the establishment of plant growth, reduce the effects of blowing sand, and assist in maintenance operations. Table 8-9 and Table 8-10 provide the side slope criteria for rural expressways and arterials, respectively. In addition, the designer should check the geotechnical report to determine the appropriate side slopes based on the local site conditions.

8.3.2.6 Right-of-Way

The right-of-way is typically configured to accommodate all of the cross-section elements throughout the project. This usually precludes a uniform right-of-way width because there are typically many situations where additional width is advantageous. Such situations occur where the side slopes extend beyond the normal right-of-way, for clear areas at the bottom of traversable slopes, for wide clear areas on the outside of curves, where greater sight distance is desirable, at intersections and junctions with other roads, at railroad-roadway grade crossings, for environmental considerations, and for maintenance and utility access.

Where additional lanes may be needed in the future, the initial right-of-way width should be wide enough to provide for the wider roadway section.

8.3.3 Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the Abu Dhabi *Road Structures Design Manual* (29).

Carry the full width of the approach roadways, including shoulders, across all new bridges.

Road overpasses, sign structures, and pedestrian bridges should have a 6.5-m clearance over the entire roadway width including the usable shoulder. For overpasses over railroads, provide a 7.5-m clearance. The designer should consider providing an additional clearance to allow for future resurfacing.

8.3.4 Intersections

The liberal use of high-type intersections and interchanges is highly desirable on arterials that do not have full control of access. Auxiliary turning lanes and adequate turning widths should generally be provided where arterials intersect other public roads. Where practical, two principal arterials that intersect should ideally be served by interchanges, possibly of the free-flow type.

Rural intersections controlled by traffic signals are normally not desirable. Drivers generally do not anticipate traffic signals in rural areas or along facilities with high-operating speeds, especially when traffic volumes are relatively low. Desirably, provide 1.5-km spacing between rural intersections.

Avoid using curbs at high-speed intersections. Curbing may present an obstacle to drivers and may become traps for blowing sand.

If interchanges are intermixed with intersections, provide adequate merging distances to allow ramp traffic to operate freely. The merging driver should not have to be concerned with cross traffic at a downstream intersection while making a merging manoeuvre. Design of intersections and interchanges should be in accordance with Chapter 10 “Intersections” and Chapter 12 “Interchanges,” respectively.

8.3.5 Access Management

Arterials are designed and constructed with the intent of providing better traffic service than is available on local and collector roads. One of the most important considerations in arterial development is the amount of access control, full or partial, that can be acquired. The ability to effectively control of access on an arterial will often reduce the frequency of access-related crashes.

The appropriate degree of access control or access management depends on the type and importance of an arterial. Anticipation of future land use is a critical factor in determining the degree of access control. Provision of access management is vital to the concept of an arterial route if it is to provide the service life for which it is designed. For additional guidance on access management, see the Abu Dhabi *Access Management Policy and Procedures Manual* (21).

8.4 Trucks Routes

Special truck routes are designed to remove trucks from major passenger car routes. These routes require wider travel lane widths, flatter grades, longer sight distances, etc., than regular roadways. Most truck routes are designed as a single carriageway; however, if traffic volumes warrant, they may be designed as a dual carriageway. Table 8-12 provides the design criteria for truck routes. Figure 8-7 illustrates a typical truck route in Abu Dhabi.

Although the maximum posted speed limit for trucks is 80 km/h, passenger cars are also allowed to travel on truck routes and are allowed go faster than 80 km/h. Consequently, the design speeds for truck routes are based on the design speed for passenger cars. Typically, the higher design speeds for passenger cars will accommodate trucks travelling at a slower speed. However, the designer should check to ensure truck stopping sight, acceleration and deceleration, and intersection sight distances are available.

Figure 8-7: Truck Route in Abu Dhabi (Route E75)

Table 8-12: Geometric Design Criteria — Rural Truck Routes

Design Element				Manual Section	Design Criteria		
Design Controls	Design Year			2.7	20 Years		
	Design Speed			2.5	Des.: 120 km/h Min.:100 km/h		
	Access Control			2.10	Full Control		
	Level of Service			2.7	Des: B Min: C		
Cross Section Elements	Travelled Way	Lane Width		3.2	Max.: 4.0 m Des.: 3.75 m Min.: 3.65 m		
		Surface Type		3.2	Paved		
	Shoulder Width	Right	Total Width	3.2	3.6 m		
			Paved Width	3.2	3.6 m		
			Surface Type	3.2	Paved		
		Left (where applicable)	Total Width	3.2	3.0 m		
			Paved Width	3.2	3.0 m		
			Surface Type	3.2	Paved		
	Auxiliary Lanes	Lane Width		3.2	3.75 m or 3.65 m		
		Shoulder Width		3.2	Desirable: Full Shoulder Width Minimum: 1.8 m		
	Cross Slope	Travel Lane		3.2	2% Typical		
		Shoulder		3.2	2% Typical		
		Auxiliary Lanes		3.2	2% Typical		
	Median Width **(1)**	Depressed		3.3	Recommend: 10 m Desirable: 18 m		
		Flush (Concrete Barrier)		3.3	7.8 m		
	Right-of-Way Width			3.6	36 m **(2)**		
Roadway Slopes	Side Slopes	Cut Section **(3)**	Front Slope	3.4	1:6		
			Ditch Width		2.4 m		
			Back Slope		1:4		
		Rock Cut		3.4	See Section 3.4		
		Fill Section	Height: 0-1 m	3.4	1:6		
			Height: 1-3 m		1:6 to Clear Zone; 1:4 to Toe		
			Height: > 3 m		1:6 to Clear Zone; 1:3 to Toe		
	Verges			3.4	1.0 m		
Bridges	Clear Roadway Width			8.3.3	Full Approach Roadway Width		
	Vertical Clearance (Truck route under overcrossing roadways, overhead signs, pedestrian bridges)			6.4	6.5 m		
	Vertical Clearance (Truck route over railroad)			6.4	7.5 m		
Alignment					**Design Speed**		
					100 km/h	**110 km/h**	**120 km/h**
	Stopping Sight Distance (Level) **(4)**			4.3	185 m	220 m	250 m
	Decision Sight Distance			4.4	315 m	330 m	360 m
	Minimum Radii	$e_{max} = 0.04$		5.2	492 m	635 m	872 m
		$e_{max} = 0.06$			756 m	561 m	756 m
	Vertical Curvature (K-values)	Crest **(5)**		6.3	52	74	95
		Sag		6.3	45	55	63
	Maximum Grade **(6)**	Level		6.2	3%	3%	3%
		Rolling		6.2	4%	4%	4%
	Minimum Grade			6.2	Desirable: 0.3% — 0.5% Minimum: 0% with proper drainage		

1. Median Width. The median width will depend upon many factors. These include the type of median (depressed or flush), the required depth of ditch, the acceptable median slopes, the available right-of-way, the anticipated ultimate development of the facility (i.e. planned addition of travel lanes), and field conditions.
2. Right-of-Way Width. The minimum ROW width will be the sum of the travel lane widths, plus the outside shoulder widths, plus the median width, plus the necessary width for fill and cut slopes and clear zones. Desirably, the ROW width will accommodate the anticipated ultimate development of the facility plus services corridors.
3. Cut Slopes. Typical values in table apply to earth cuts; see Section 3.4 for rock cuts.
4. Stopping Sight Distances (SSD). SSD provided are for passenger cars. The designer should ensure truck SSD is available assuming the truck design speed and height of eye. See Section 4.3.3 for truck stopping sight distances.
5. Crest Vertical Curve K-Values. K-values were developed using the SSD passenger cars. The designer should ensure truck SSD is available assuming the truck design speed and height of eye.
6. Maximum Grade. With wide medians where two roadways are on independent alignments, downgrades may be 1% steeper. Grades in mountainous terrain may be 1% steeper.

9 URBAN STREETS

Urban streets are all streets within the existing and planned urban areas of the Emirate of Abu Dhabi (as shown in the Surface Transport Master Plans). Urban areas are defined in the maps contained in Plan Abu Dhabi 2030, Plan Al Ain 2030, and Plan Al Gharbia 2030. Where ambiguity arises regarding the jurisdiction of a particular street (e.g. urban streets transitioning to rural roads), consult with the proper authority for guidance.

9.1 Overview

Chapter 9 provides guidance in the design of urban streets including design speed, cross sections, medians, low-speed horizontal alignment, frontage lanes, on-street parking, and pedestrians. Information that is also applicable to these facilities is included in the following sources:

- Chapter 2 “General Design Criteria and Controls,” Chapter 3 “Cross Section Elements,” Chapter 4 “Sight Distance,” Chapter 5 “Horizontal Alignment,” and Chapter 6 “Vertical Alignment” provide guidance on the geometric design elements that are also applicable to these facilities.

- Chapter 10 “Intersections” provides information on the design of intersections, including left- and right-turn lanes, and channelization.

- Chapter 11 “Roundabouts” provides guidance on the design of roundabouts.

- Abu Dhabi Urban Planning Council *Abu Dhabi Urban Street Design Manual* (4) provides urban street design criteria for the municipalities of the Emirate of Abu Dhabi. It also provides design guidance on retrofitting urban streets and intersections/junctions.

- The ITE *Urban Street: Geometric Design Handbook* (42) provides guidance on the operational and safety aspects associated with the geometric design of urban roads and streets.

- Abu Dhabi *Roadside Design Guide* (28) provides guidelines on roadside safety issues.

- Abu Dhabi Public Transport Division *Public Transport Bus Service Planning Guidelines* addresses public transportation issues (43).

9.2 General

9.2.1 Urban Design Challenges

Urban street design involves a variety of issues that are not a factor in rural designs. Closely spaced intersections that have wide fluctuation in traffic speeds and volume inheritably make urban street designs more complex. Some of the challenges and situation frequently encountered by the designer of urban streets include:

- insufficient right-of-way for all desired cross section elements;
- constrained vertical and horizontal alignment;
- limited sight distance;
- narrow lane widths;

- numerous access drives to private property;
- speed control in residential areas;
- presence of pedestrian and cyclists;
- accommodation of disabled pedestrians;
- accommodation of numerous utilities;
- at-grade road/railroad grade crossings;
- accommodation of bus and rail transit service;
- specialised design in sensitive areas (e.g. mosques, parks, activity centres);
- emphasis on aesthetic integration with adjacent properties;
- growing public desire to balance societal goals and community values with transportation facility designs; and
- accommodation of taxis.

9.2.2 Urban Street Typology

9.2.2.1 Functional Classification

AASHTO functionally classifies urban streets as arterials, collectors, and local roads based on the guidance provided in the AASHTO *A Policy on Geometric Design of Highways and Streets* (1). The Abu Dhabi Urban Planning Council in its *Abu Dhabi Urban Street Design Manual* (4) defines urban streets with a two-name convention. The first name is the context name, based on land use and community character (see Section 9.2.2.2) and the second name is the street family name (i.e. boulevards, avenues, streets, and access lanes), describing the transport capacity of the street. Section 2.4.4 provides the definitions for the DMT urban street family names. Table 2-5 shows the relationship between the DMT street family classification and the AASHTO functional classification systems. The design criteria in this chapter are based on the DMT street family classification. However, the criteria presented elsewhere in this *Manual* is based on AASHTO functional classification.

9.2.2.2 Street Context Names

DMT street context names and characteristics are defined below:

1. <u>City</u>. The central business district and high-density mixed-use neighbourhoods around the city are generally characterised by high levels of activity. In large cities, some district centres may also have these characteristics.

2. <u>Town</u>. Mixed use areas with medium levels of pedestrian activity, where buildings are typically three to five stories.

3. <u>Commercial</u>. Areas throughout the city intended to provide a variety of working, shopping, and service options as well as convenience.

4. <u>Residential Neighbourhood</u>. Areas that provide a variety of housing opportunities, with densities varying from villa to multi-dwelling residential buildings. An Emirati neighbourhood is a sub-set of the residential context, primarily designed for the very low-density Emirati neighbourhoods comprising only of villas.

5. Industrial. Areas for businesses that include production, warehousing, and distribution with support commercial services, ancillary office space, and labour housing. These areas generally have low levels of pedestrian activity and limited active frontage.

9.2.3 Access Management

Some type of access control is desirable on urban roads and streets. The principal reason for placing controls on access along the edge of urban streets is to protect the functional integrity of the boulevard or avenue and, to the lesser extent, the urban street. While access to abutting property is usually required, it should be carefully regulated to limit the number of access points and their locations. Both safety and capacity of the urban streets are negatively affected by access points. The Abu Dhabi *Access Management Policy and Procedures Manual* (21) provides guidance in control access along urban streets.

9.2.4 Design Priorities

In line with the goals of the Plan Abu Dhabi 2030 to promote walking and the creation of a sustainable urban environment, the urban street design priorities are:

1. First Priority – Pedestrians. All streets must be safe and comfortable for pedestrians of all ages and abilities.

2. Second Priority – Transit Users. Users of transit facilities are among the most efficient users of street space. These facilities include Metro, Light Rail Transit (LRT), Bus Rapid Transit (BRT), and buses.

3. Third Priority – Cyclists. Cyclists are vulnerable users, and their safety must be considered during design. They are also among the most efficient users of street space.

4. Fourth Priority – Motor Vehicles. The accommodation of motor vehicle traffic is important to the continuing growth of the Emirate of Abu Dhabi. However, when considering traffic accommodation on urban streets, it is essential that the nondriving options be at least as attractive as those that involve the use of private motor vehicles.

User priorities listed here are differentiated from functional or operational priorities as this will vary depending on the street type, anticipated travel demand and land use context.

9.3 General Design Elements

9.3.1 Design Speed

The design tables in Section 9.5 provide the design speed criteria for urban streets based on the street types. In relatively undeveloped locations and where economics, environmental conditions, and signal spacing permits, consider using higher design speeds.

9.3.2 Capacity Analyses

Transportation volumes and characteristics of all users to be served are major factors in determining the geometric criteria for urban streets. These characteristics are determined by pedestrian demands, transit users, cyclists, and motor vehicular users. For urban streets, the designer needs to determine the average daily traffic (ADT), peak-hour volumes, peak-hour factor, directional distribution, traffic composition, and projection of future transportation demands for all modes of travel. See the *Highway Capacity Manual 2010* (HCM) (17) for guidance on making these determinations. The design volumes should be estimated for 20 years after construction.

In selecting the appropriate level of service for the facility, compromises may be required by the various users. Where compromises are required, the designer should consider the design priorities as noted in Section 9.2.4.

9.3.2.1 Analyses

For the purpose of analysis, urban streets are separated into individual elements that are physically adjacent and operate as a single entity. A point represents the boundary between links, which is represented by an intersection or ramp terminal. A link represents a length of roadway between two points. A link and its boundary points are referred to as a segment. Multiple contiguous segments can be combined into a single facility.

Table 9-1 identifies the common performance measurements for non-automobile modes. Table 9-2 identifies the input data needed for the determining capacity analysis for the automobile mode on urban streets. For the subject travel direction, these elements must be provided for each segment and for the through-movement group at each boundary intersection.

9.3.2.2 Pedestrian Level of Service

To accommodate different pedestrian activities (walking and waiting), two approaches are used to calculate pedestrian level of service (LOS):

<u>Walking LOS (Pedestrian Walking Areas).</u> The first method is applicable to the through zone at a mid-block location, where there are no crossings or junctions located. It is based on available space (m2) per pedestrian. To calculate the Walking LOS, a snapshot is taken of the street and the number of pedestrians is counted within a specific area. The sample area, in m^2, is divided by the number of pedestrians to get a value. This value is evaluated with the criteria in Table 9.3 to obtain the walking LOS.

For existing streets, a survey of pedestrian counts will be required. For new streets, an estimate of projected pedestrian counts will be required, based on the street context and adjacent land uses. For additional information regarding calculation of LOS based on flow rate, platooning, etc., see the *HCM* (17).

<u>Waiting LOS (Pedestrian Queuing Areas)</u>. The second method, Waiting LOS, is applicable at junctions/intersections and other crossings and is based on the available area (m^2) per pedestrian. The available area is defined as the waiting area directly adjacent to the crossing. There are typically three types of areas for pedestrian waiting:

- sidewalk corner before a crossing,
- refuge island at right turn slip lane, and
- median refuge island.

The limiting criteria in these cases will be either the splitter island or the median refuge. The waiting area is to be calculated based on the available flat, level waiting space. However, areas inclined at a slope of less than 1:12 (8%) can be included in waiting areas.

Table 9-1: Urban Streets Performance Measures

Mode	Performance Measures
Pedestrian	Sidewalk crowding Average crossing delay including average distance to crossing Frequency of protected crossings Percentage of active building edge along sidewalk Percentage of shaded sidewalk Average block perimeter
Transit Users	Junction/intersection delay Corridor travel time relative to speed limit Passenger crowding Reliability Frequency Service hours Thermal comfort at waiting areas
Cyclists	Presence of cycle lane or cycle track Cycle level of service

Source: (4)

Table 9-2: Automobile Input Data Requirements

Data Category	Location	Input Data Element	Basis
Geometric Design	Segment	Segment length	Segment
Other	Segment	Analysis period duration	Facility
Performance Measures	Boundary Intersection	Volume-to-capacity ratio	Through-movement group
	Segment	Base free-flow speed Travel speed	Segment Segment

Source: (17)

Table 9-3: Pedestrian LOS

Level of Service	Walking LOS Space per Pedestrian (m^2/ped)	Waiting LOS Average Pedestrian Area (m^2/ped)
A	> 3.3	> 1.2
B	2.3 – 3.3	0.9 – 1.2
C	1.4 – 2.3	0.6 – 0.9
D	0.9 – 1.4	0.3 – 0.6
E	0.5 – 0.9	0.2 – 0.3
F	< 0.5	< 0.2

Source: (4)

Table 9.3 should be used during the design process to estimate the pedestrian LOS. The parameters need to be based on the design cross section and pedestrian counts taken from the pedestrian survey and an estimation of any anticipated increase in pedestrian activity, based on the improvements to be undertaken. Table 9-4 provides the parameters for each pedestrian LOS:

If pedestrian crossing counts are used, then use the actual number of pedestrians crossing in both directions during a crossing phase.

There are a number of other calculation methods for determining pedestrian LOS for urban streets. For example, the HCM includes a schedule of pedestrian LOS values, which considers flow rate and speed. Consult with the Client to ensure the correct method is adopted for specific cases where pedestrian LOS is required for a transportation impact study.

Table 9-4: Pedestrian Walking LOS Descriptions

LOS	Description
A	At LOS A, pedestrians move in desired paths without altering their movements in response to other pedestrians. Walking speeds are freely selected, and conflicts between pedestrians are unlikely.
B	At LOS B, there is sufficient area for pedestrians to select walking speeds freely to bypass other pedestrians and to avoid crossing conflicts. At this level, pedestrians begin to be aware of other pedestrians, and to respond to their presence when electing a walking path.
C	At LOS C, space is sufficient for normal walking speeds, and for bypassing other pedestrians in primarily unidirectional streams. Reverse-direction or crossing movements can cause minor conflicts, and speeds and flow rate are somewhat lower.
D	At LOS D, freedom to select individual walking speed and to bypass other pedestrians is restricted. Crossing or reverse-flow movements face a high probability of conflict, requiring frequent changes in speed and position. The LOS provides reasonably fluid flow, but friction and interaction between pedestrians is likely.
E	At LOS E, virtually all pedestrians restrict their normal walking speed, frequently adjusting their gait. At the lower range, forward movement is possible only by shuffling. Space is not sufficient for passing slower pedestrians. Cross- or reverse-flow movements are extremely difficult. Design volumes approach the limit of walkway capacity, with stoppages and interruptions to flow.
F	At LOS F, all walking speeds are severely restricted, and forward progress is made only by shuffling. There is frequent unavoidable contact with other pedestrians. Cross-and reverse-flow movements are virtually impossible. Flow is sporadic and unstable. Space is more characteristic of queued pedestrians than of moving pedestrian streams.

Source: (4)

9.3.2.3 Public Transportation Services

The LOS definitions for public transport services are given in Table 9-5. The LOS is based on service frequency, headway, and temporal coverage of the public transport services. LOS B should be considered acceptable for planning and operational analysis using any of the measures of performance criteria.

Table 9-5: LOS Definitions for Public Transport Services

Level of Service	Measures of Performance			
	Service Frequency (veh/h)	Headway (minutes)	Span of Service (hrs)	Comments
A	> 6.0	< 10	19 – 24	Passengers do not need schedules
B	4.01 – 6.0	10 – 14	17 – 18	Frequent Service, passengers consult schedules
C	3.0 – 4.0	15 – 20	14 – 16	Maximum desirable time to wait if transit vehicle missed
D	2.0 – 2.99	21 – 30	12 – 13	Service unattractive to choice riders
E	1.0 – 1.99	31 – 60	4 – 11	Service available during hour
F	< 1.0	> 60	0 – 3	Service unattractive to all riders

Source: (18)

9.3.2.4 Cycle Facilities

The LOS definitions for cycle intersections and roundabouts are given in Table 9-6. The capacity of a cycle facility depends on the number of effective lanes used by cyclists, which is found to be more important than the total width of the cycle facility or of the individual lanes. Different sets of LOS criteria have been developed based on the type of cycle facility and the performance measures criteria include speed, delay at signalised intersections and percentage of hindrance. LOS A should be considered acceptable for planning and operational analysis using any of the measures of performance criteria.

Table 9-6: LOS Definitions for Cycle Facilities

Level of Service	Measures of Performance		
	Delay at Signalised Intersections(s)	Uninterrupted Flow (% of Hindrance)	Speed on Urban Street (km/h)
A	< 10	< 10	22
B	> 10 – 20	> 10 – 20	14 – 22
C	> 20 – 30	> 20 – 40	11 – 14
D	> 30 – 40	> 40 – 70	8 – 11
E	> 40 – 60	> 70 – 100	6 – 8
F	> 60	100	< 6

Source: (18)

9.3.2.5 Vehicular Level of Service

Through-vehicle travel speed is used to characterise vehicular LOS for a given direction of travel along an urban street facility. This speed reflects the factors that influence running time along each link and the delay incurred by through vehicles at each boundary intersection. This performance measure indicates the degree of mobility provided by the facility. The following characterise each service level:

1. LOS A. LOS A describes primarily free-flow operation. Vehicles are completely unimpeded in their ability to manoeuvre within the traffic stream. Control delay at the boundary intersections is minimal. The travel speed exceeds 85% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

2. LOS B. LOS B describes reasonably unimpeded operation. The ability to manoeuvre with the traffic stream is only slightly restricted and control delay at the boundary intersections is not significant. The travel speed is between 67% and 85% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

3. LOS C. LOS C describes stable operation. The ability to manoeuvre and change lanes at midsegment locations may be more restricted than at LOS B. Longer queues at the boundary intersections may contribute to lower travel speeds. The travel speed is between 50% and 67% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

4. LOS D. LOS D indicates a less stable condition in which small increases in flow may cause substantial increases in delay and decreases in travel speed. This operation may be due to adverse signal progression, high volume, or inappropriate signal timing at the boundary intersections. The travel speed is between 40% and 50% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

5. LOS E. LOS E is characterised by unstable operation and significant delay. Such operations may be due to some combination of adverse progression, high volume, and inappropriate signal timing at the boundary intersections. The travel speed is between 30% and 40% of the base free-flow speed and the volume-to-capacity ratio is no greater than 1.0.

6. LOS F. LOS F is characterised by flow at extremely low speed. Congestion is likely occurring at the boundary intersections, as indicated by high delay and extensive queuing. The travel speed is 30% or less of the base free-flow speed. Also, LOS F is assigned to the subject direction of travel if the through movement at one or more boundary intersections has a volume-to-capacity ratio greater than 1.0.

Table 9-7 lists the LOS thresholds established for automobiles on urban streets. Table 9-8 provides the maximum number of lanes for each street family classification.

Table 9-7: Urban Streets LOS

Level of Service	Measure of Performance				
	Average Speed (km/h)				Volume/Capacity Ratio
A	> 72	> 59	> 50	> 41	0 – 0.6
B	56 – 72	46 – 59	39 – 50	32 – 41	0.6 – 0.7
C	40 – 56	33 – 46	28 – 39	23 – 32	0.7 – 0.8
D	32 – 40	26 – 33	22 – 28	18 – 23	0.8 – 0.9
E	26 – 32	21 – 26	17 – 22	14 – 18	0.9 – 1.0
F	< 26	< 21	< 17	< 14	> 1

Source: (18)

Table 9-8: Number of Travel Lanes for Urban Streets

Street Family	Number of Travel Lanes
Boulevard	3 + 3*
Avenue	2 + 2
Street	1 + 1
Access Lane	1 + 1 1 shared

**Boulevards may be 4+4 where existing or where necessary based on travel demand.*

9.3.3 Sight Distance

Sight distance is directly related to and varies appreciably with design speed. Stopping sight distance should be provided throughout the length of the roadway.

Providing decision sight distance at locations where complex decisions are made greatly enhances the capability for drivers to safely accomplish manoeuvres. Example locations where complex decisions may be necessary include signalised intersections (junctions), transitions in roadway widths, lane shifts, roundabouts, merge/diverge locations, and transitions in the number of lanes (e.g. turn lanes).

At stopped-controlled intersections, sight distance is required to allow the stopped motorist to safely turn left, right, or cross the major street. At signalised intersections, provide sufficient sight distance so that the first vehicle on each approach is visible to all other approaches.

See Chapter 4 “Sight Distance” for a comprehensive discussion of stopping, decision, and intersection sight distances.

9.3.4 Alignment

The alignment should closely fit with the existing topography to minimise the need for cuts or fills. Roadway alignment in commercial and industrial areas should be commensurate with the topography, but should be as direct as practical.

Section 9.4 discusses the design process for horizontal alignment on low-speed, urban streets (i.e. ≤ 70 km/h). For urban streets with design speeds equal to or greater than 80 km/h, the designer should use the horizontal alignment criteria as discussed in Chapter 5 “Horizontal Alignment.”

Chapter 6 “Vertical Alignment” provides guidance on vertical alignment, which is applicable to urban streets.

9.3.5 Grades

The length and steepness of grades directly affect the operational characteristics of a road. Steep grades affect truck performance, intersection operations, and the capacity of the facility. The design tables in Section 9.5 provide the maximum grade criteria for urban streets based on street classification, area type, and terrain.

For curbed facilities, it is necessary to provide a minimum longitudinal grade of 0.3% to facilitate drainage. Desirably, the longitudinal grade should be 0.5%. For uncurbed facilities, a minimum longitudinal grade of 0.0% may be considered if adequate cross slopes are provided and there are no plans to add curbing in the future.

9.3.6 Cross Section Elements

The designer needs to balance the needs of pedestrians, cyclists, parked vehicles, buses, trucks, and passenger vehicles within the available right-of-way. In areas with restricted right-of-way, compromises may be required between reducing lane widths, removing on-street parking, removing or narrowing medians, and/or reducing pedestrian realms. The *Abu Dhabi Urban Street Design Manual* (4) provides guidance where there is too little or too much right-of-way available. Figure 9-1 through Figure 9-5 present typical city cross sections. See the *Abu Dhabi Urban Street Design Manual* (4) for typical cross sections for town, commercial, residential/Emirate neighbourhood, and industrial contexts. The designer may use the Abu Dhabi Urban Street Design Manual Online Design Tool (available on the Urban Planning Council's website) as a simple tool to quickly design cross-sections in urban areas.

9.3.6.1 Street and Shoulder Widths

Travel lane widths on urban streets will vary according to the street typography. See the design tables in Section 9.5.

Shoulders/shy distances are generally not used on urban streets. However, shy distances may be used for urban arterials if considered beneficial. For non-curbed facilities, the designer should consider including shoulders for pavement support, safety, etc. Cross Slopes

Roadway cross slopes provide for proper drainage. For single carriageways, provide a uniform cross slope of 1.5 to 2% from one curb line to the other. For dual carriageways, provide a uniform 1.5 to 2% cross slope across the travelled way sloping away from the median.

9.3.6.2 Curbing

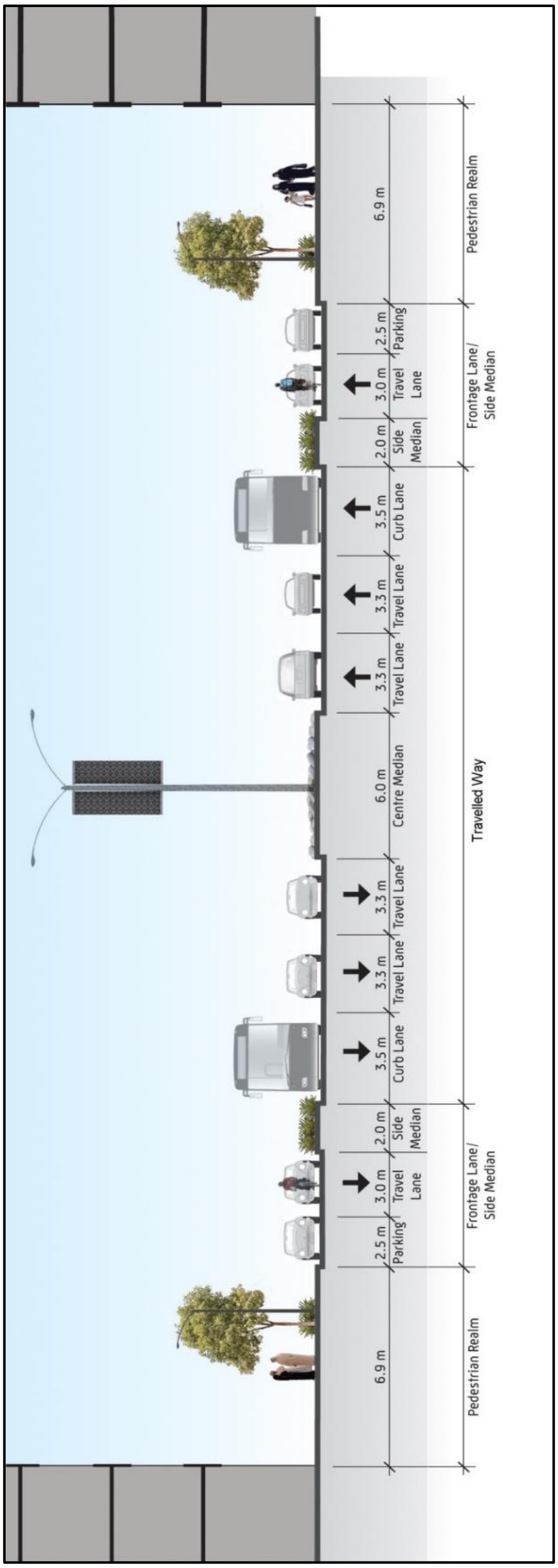
Urban streets are normally designed with curbs to allow greater use of the available width, control of drainage, protection of pedestrians, access control, and delineation. On urban boulevards, avenues, streets, access lanes, frontage lanes and parking areas, 100 mm high upstand curb is typically used. Higher curbs may be installed at transit stops.

Where parking in a pedestrian realm is an issue, use pedestrian protection techniques (e.g. bollards, planters) instead of higher curbs.

9.3.6.3 Parking

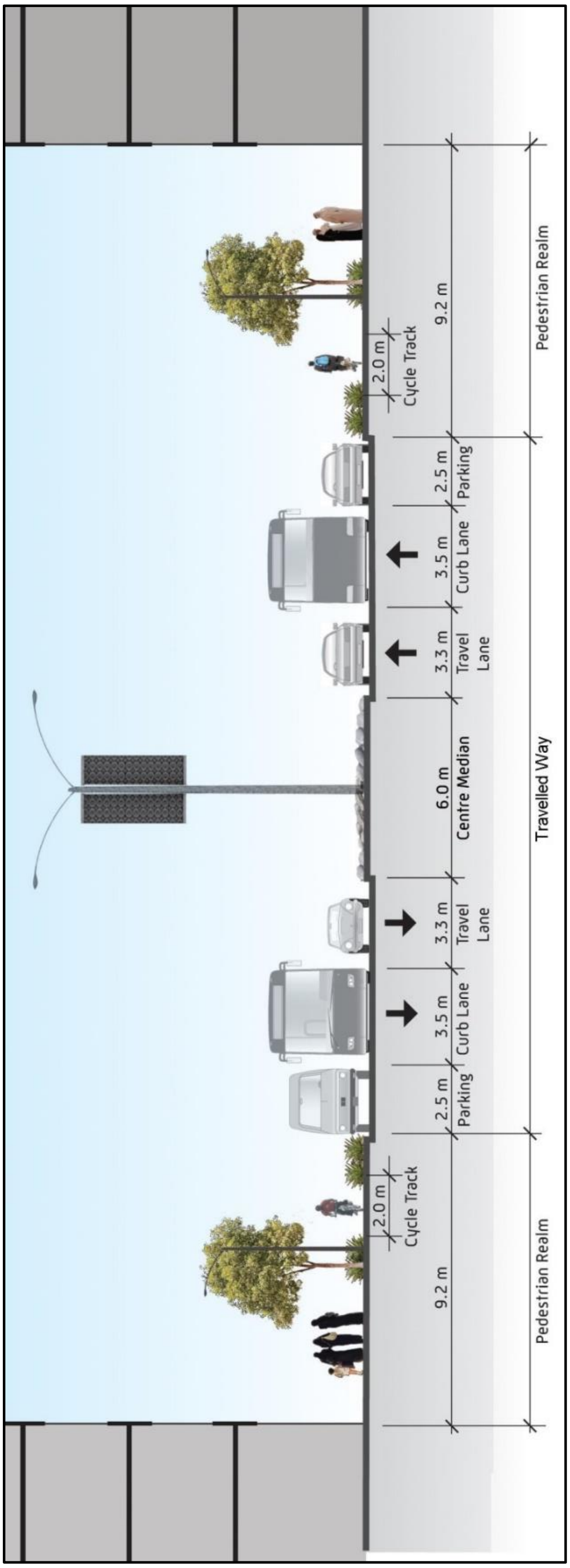
For most urban projects, the designer must evaluate the demand for parking. Desirably, these parking needs will be accommodated by providing off-street parking facilities or frontage areas; see Figure 9-6. Section 16.9 provides guidance on the design and layout of off-street parking facilities.

Figure 9-1: Typical City Boulevard (with Frontage Lane)



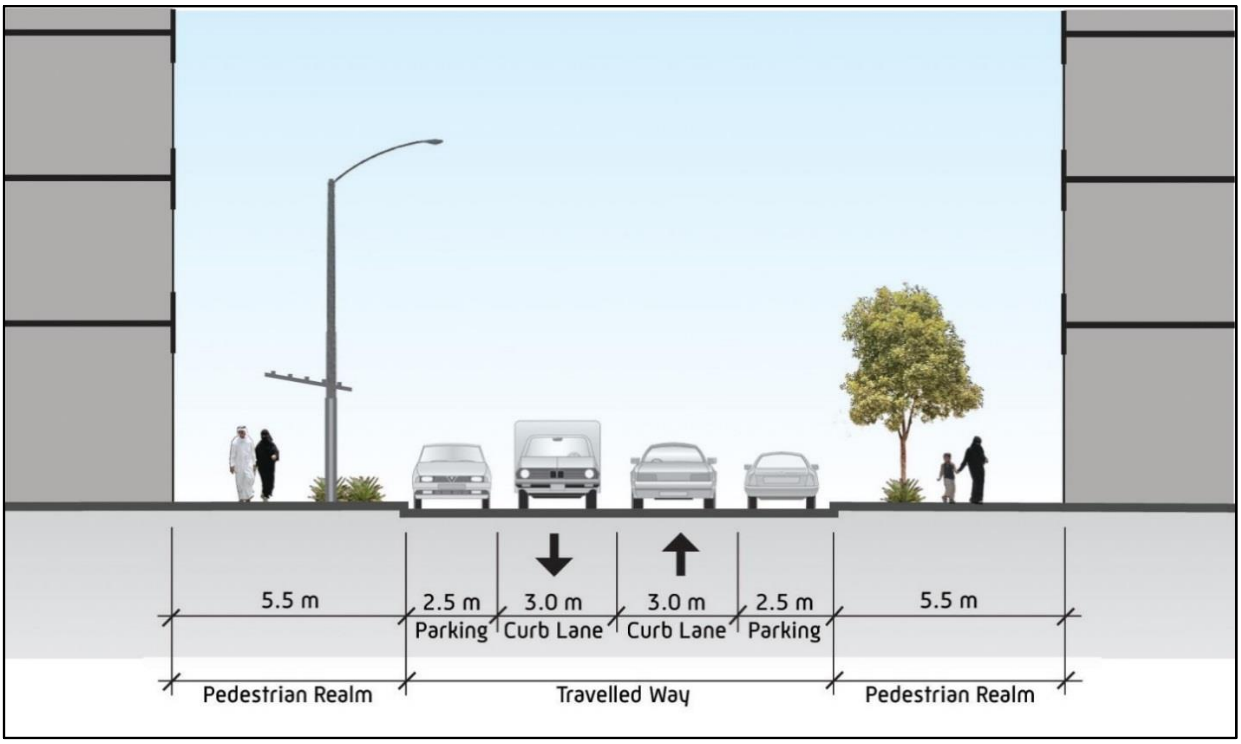
Source: (4)

Figure 9-2: Typical City Avenue



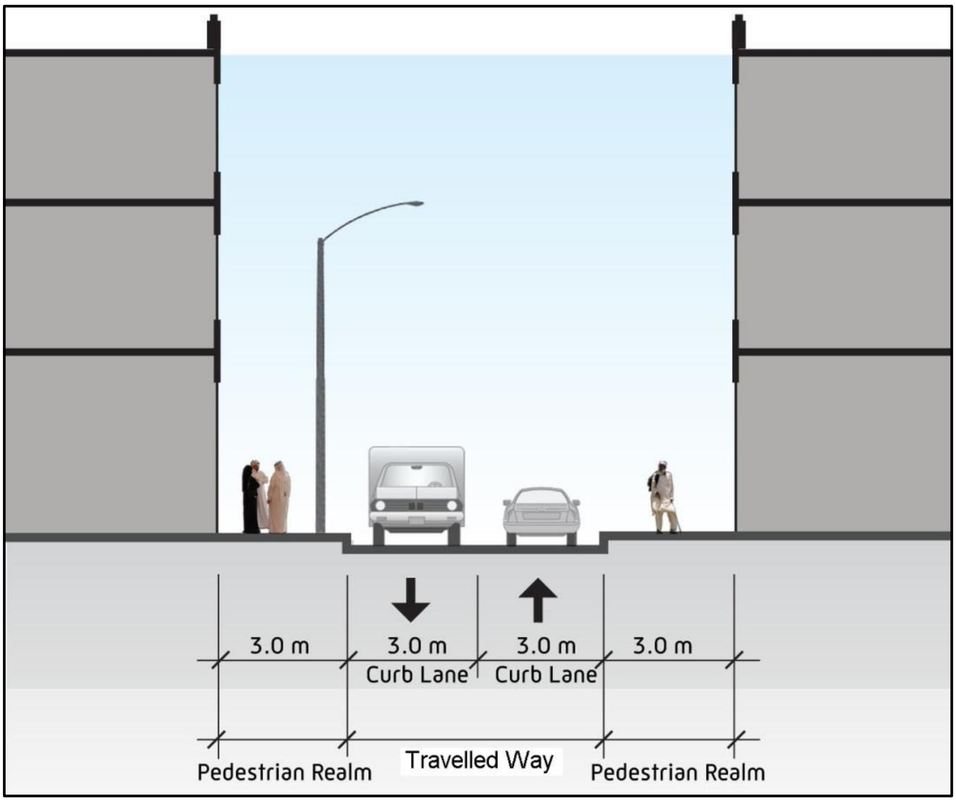
Source: (4)

Figure 9-3: Typical City Street



Source: (4)

Figure 9-4: Typical City Access Lane



Source: (4)

Figure 9-5: Typical City Boulevard

Figure 9-6: Typical City Boulevard with Frontage Road

When providing on-street parking along urban streets, the designer should evaluate the following:

1. Warrants. Boulevards may only provide on-street parking through the addition of a frontage road. On-street parking is typically provided on avenues, streets, and access lanes.

2. Configurations. There are two basic types of parking — parallel and angle parking. These are illustrated in Section 16.9. On-street parking should generally be parallel parking.

3. Widths. The design tables in Section 9.5 provide guidance for parallel parking lane widths. See Section 16.9 for off-street parking stall dimensions. Edge zone must be a minimum of 1.5 m where there is on-street parking or a cycle track. The edge zone may be 0.20 m where there is sufficient room available for signing, lighting, and utilities within an adjacent furnishings zone.

4. Angled Parking. For 1-way local roads, access roads and parking areas, diagonal or parallel parking is preferred whereas perpendicular parking may also be applied in special cases, such as traffic circulation and parking demand; however, road safety issues shall also be taken into consideration. For 2-way local roads, access roads and parking areas perpendicular and parallel parking shall be applied.. Also, the designer should consider the following:

   a. Angle. Desirably, the parking angle should be 45° to 60°. Adjacent to playgrounds, sidewalk cafes, vendors, etc., 90° parking may be acceptable depending upon traffic volumes and speeds.

   b. Backing Manoeuvre. Angle parking requires the driver to back into the travelled way when sight distance may be restricted by adjacent parked vehicles. This manoeuvre may surprise an approaching motorist. The parked car will require a certain distance to back out of its stall. Whether or not this is a reasonably safe manoeuvre will depend upon the number of lanes in each direction, lane widths, operating speeds, traffic volumes during peak hours, parking demand, and turnover rate of parked vehicles.

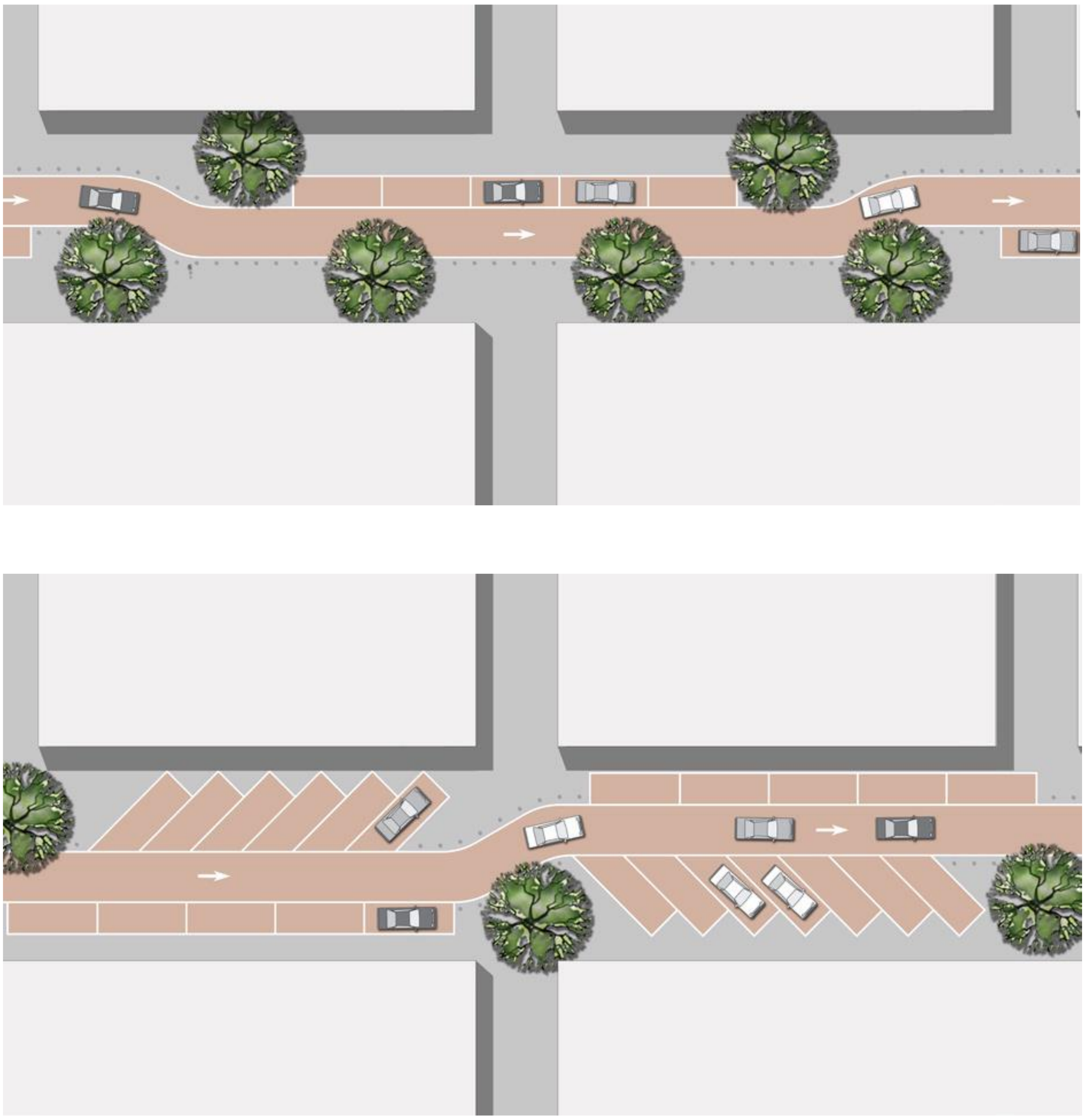
   c. Reverse Angle Parking. On very low-volume and low-speed urban streets, reverse angle parking may be acceptable. Reverse angle parking enhances visibility between motorists and cyclists. If used, provide adequate signage and educate the local motorists.

   d. Crashes. After analysing backing manoeuvre space, angle parking may be considered to remain if there is no history of crashes relating to the existing angle parking.

5. Stall Dimensions. The minimum dimensions for parking stalls can be found in the Abu Dhabi *Traffic Impact Study Guidelines* (18) and in Section 16.9.

6. Traffic Calming. Blocks of parking should be used to divert vehicular routes and provide traffic calming. These should not be longer than 4 to 6 parallel parking bays; see Figure 9-7.

Figure 9-7: Parking Used as Traffic Calming



7. Location. When locating parking spaces, the designer should consider the following:

    - Prohibit parking within 6.0 m of any crosswalk.
    - Prohibit parking within 3 m to 5 m of the beginning of the curb radius at mid-block driveway entrances.
    - Prohibit parking within 15 m of the nearest rail of road/railroad crossing.

- Prohibit parking within 10.0 m of the beginning of the curb radius on the leg to any intersection with a Yield control, flashing beacon, stop sign.

  - In addition prohibit parking within the approach and exit to a Traffic Signal due to visibility to Pedestrians and Primary Signal Heads and maneuvering difficultiesProhibit parking on bridges or within a roadway tunnel.

  - Prohibit parking from areas designated by local traffic and enforcement regulations (e.g. near school zones, loading zones). See local ordinances for additional information on parking restrictions.

  - Prohibit parking at designated bus stops.

  - Check intersection sight distance to side roads.

- Eliminate parking across from a T-intersection. Where Parking demand is high, parallel parking may be allowed as long as it does not affect visibility on both approach and exit;

9.3.6.4 Medians

Section 3.3 discusses the various medians that may be used in urban areas and guidelines for selecting medians and widths. In addition, the designer should consider the following for medians on urban streets:

1. Flush Medians. These median types may be used in both constrained and open urban areas. Median widths may be 1.2 m to 3.0 m. Note that flush medians are unacceptable where pedestrian refuges are necessary.

2. Depressed Medians. In open urban areas, a depressed median may be used. This design is typically used with left shoulders and where the design speed is 80 km/h or greater. Sections 3.3 and 8.3.2 provide further guidance on depressed medians.

3. Raised Medians. Usually, a raised median is proposed in urban areas where managed access to the road and control of left-turn movements are desired. Section 3.3 provides guidance on the selection and design of raised medians. Chapter 10 “Intersections” illustrates typical treatments for left-turn lanes with raised medians. The designer should note the following:

   a. Dual-Left Turns. Where there is a need for dual left turns, the minimum width of the median is 9.5 m. Where major intersections are closely spaced and there is a need for dual lefts at most intersections, provide a 9.5 m median width along the entire road.

   b. Concentrated Left-Turn Movements. With raised medians, left-turn movements are concentrated at the intersections, thereby reducing the overall conflict areas of the facility. However, drivers are forced to make all left turns at the intersections, which may overload the capacity of the intersections, increase driver travel time, and create the need for U-turns at intersections.

   c. Pedestrian & Cyle Refuge. A minimum width of 2.0 m is required for a pedestrian refuge. Where there is heavy pedestrian activity or a cycle route, provide a minimum median width of 3.0 m.

d. Pedestrian Fencing. Pedestrian fencing may be desirable in the median to stop pedestrian crossings outside of designated locations. See Section 16.5.3 for additional guidance on pedestrian fencing.

e. Businesses. Business owners may perceive an adverse effect when a raised median is proposed. See NCHRP 395 *Capacity and Operations Effects of Midblock Left-Turn Lanes* (44) for guidance on the effects of raised medians.

9.3.6.5 Pedestrian Realms

Pedestrian realms are integral parts of the urban environment. In urban areas, travellers frequently choose to make trips on foot, and pedestrians desire to use a paved surfaced for these trips. Special care and consideration is required to protect pedestrians. Section 13.2, the *Abu Dhabi Urban Street Design Manual* (4), and the *Abu Dhabi Public Realm Design Manual* (13) provide guidance on the design criteria for pedestrians.

9.3.6.6 Public Transit Facilities

Transit systems (e.g. Metro, tram, buses BRT, taxis) provided optimal service when streets are designed to accommodate transit vehicles and stops. The designer should review the Abu Dhabi *Public Transport Bus Service Planning Guidelines* (43) for the design of transit systems.

Bus stop locations can be categorised as far-side, near-side, and midblock stops. Bus stops may be designed with a bus bay or pull out to allow buses to pick up and discharge passengers in an area outside of the travel lane; see Figure 9-8. This design feature allows traffic to flow freely without the obstruction of stopped buses.

Figure 9-8: Bus Turnout

9.3.6.7 Cycle Facilities

Cycles provide an extremely efficient and sustainable means of transportation. Designs of new and existing urban streets in Abu Dhabi are required accommodate cyclists in accordance with the provisions of Abu Dhabi *Walking and Cycling Master Plan* (23).

Cycle facilities may be provided in the pedestrian realm in the form of cycle tracks, or within the travelled way as cycle lanes or shared lanes. Cycle tracks and cycle lanes are dedicated cycle paths, whereas shared lanes are not delineated and share the travel lanes with motorised vehicles.

Cycle facility selection is largely a function of motor vehicular speed and volume, plus available right-of-way width; cycle facilities may vary along a route meet the site conditions.

The Abu Dhabi *Walking and Cycling Master Plan* (23) and *Abu Dhabi Urban Street Design Manual* (4) provide guidance on the design criteria for cycle facilities.

All Cycle Facilities designs or schemes must be subject to Road Safety Audits Stage- 1 (at Preliminary Design) and Stage-2 (at Detailed Design) followed by Stage-3 (Pre-opening) in accordance with TR-540; the focus should be on NMU safety, positive protection/separation from roadway traffic and speed calming features along the cycle route.

9.3.6.8 Roadside

The Abu Dhabi *Roadside Design Guide* (28) provides the minimum clear zone distances based on the design speed, traffic volumes, and side slopes. The clear zone should be clear of all unyielding objects (e.g. trees, non-breakaway sign supports, utility poles, luminaries) and other fixed objects. In urban areas, this is often very difficult to achieve. The designer should strive to provide a 0.5-m to 1.5-m lateral offset from the face of the curb to all roadside elements within the pedestrian realm.

Traffic barriers are rarely used in urban area. They often restrict access for pedestrians and cyclists. At low speeds, traffic barriers may have a similar cash severity potential as the object being protected. For additional guidance on traffic barriers, see the Abu Dhabi *Roadside Design Guide* (28).

9.3.6.9 Service Roads

In general, service roads (frontage lanes) are parallel to the travelled way, may be provided on one or both side of the roadway, and may or may not be continuous. Although they are used by motor vehicles, frontage lanes are considered part of the pedestrian realm. The designer should consider the following:

1. Location. Frontage lanes are required where there is parking demand on boulevards; they may also be included on avenues; see Figure 9-6.

2. Speed. It is essential to keep frontage lanes as narrow as possible in order to ensure slow travel speeds.

3. Emergency Vehicles. Frontage lanes do not need to accommodate fire trucks. Fire trucks can access buildings from the boulevard’s main travel lanes.

4. Angle Parking. Avoid diagonal and perpendicular parking along frontage lanes. Angle parking requires extra width for the drive aisle and, therefore, encourages excessive speeds. Only consider using diagonal and perpendicular parking in existing situations where significant right-of-way is available and there is a shortage of other available parking.

5. Cyclists. Frontage lanes may include a shared lane for cycles in the same direction of traffic. Additional cycle track may be needed for the opposite direction for cyclists. Cyclists should not be riding against vehicular traffic on the frontage lanes

6. Intersections. Avoid connections to frontage lanes except at intersecting streets. Cars entering a street from a frontage lane should be required to stop and yield to traffic on the street.

7. Taxis/Transit Stops. At taxi lay-bys and bus stops, eliminate on-street parking along a frontage lane and/or deviate the frontage lane into the furnishings zone in order to provide sufficient waiting area for transit and taxi passengers along the side median.

8. Raised Speed Tables. Use raised intersections and speed tables where access lanes or major pedestrian paths intersect frontage lanes in order to reduce speeds.

9. Design. Frontage lane entry and exit designs are determined on a site-by-site basis.

10. Side Median. The area between the travelled way of a through-traffic roadway and a frontage lane or street is referred to as the side median or outer separation. Side medians function as buffers between the through traffic and the local traffic on the frontage lane and provide refuge for crossing pedestrians. Side median should provide adequate buffer and lateral clearance for street furniture on both sides of the travelled lanes.

9.3.6.10 Right-of-Way

The width of right-of-way for the complete development of an urban street is influenced by both vehicular and non-motorised traffic demands, topography, land use, cost, intersection design, and the extent of ultimate expansion. The width of right-of-way should be the summation of the various cross-sectional elements: through travelled ways, parking lanes, cycle lanes, medians, auxiliary lanes, pedestrian realms, and, where appropriate, frontage roads, roadside clear zones, side slopes, drainage facilities, utility appurtenances, and retaining walls.

The right-of-way width should be based on the preferable dimensions of each element to the extent practical in developed areas. However, the designer is often confronted with the problem of providing an overall cross section that will give the maximum service within the available right-of-way. Right-of-way widths in urban areas are governed primarily by economic considerations, physical obstructions, and/or environmental concerns. Along any urban route, conditions of development and terrain vary, and accordingly, the availability of right-of-way varies. For this reason, the right-of-way on a given facility should not be a fixed width predetermined based on the most critical point along the facility. Instead, a desirable right-of-way width should be provided along most, if not all, of the facility.

When redesigning existing streets, addressing safety and security requirements, or accommodating utilities for new or existing streets, the total right-of-way dimensions may not meet the typical street design dimensions. The *Abu Dhabi Urban Street Design Manual* (4) offers guidance for where there is not enough right-of-way and where there is excessive right-of-way.

9.3.7 Traffic Control Devices

Traffic control devices should be applied consistently and uniformly. Details of the standard devices and warrants for various conditions are found in the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

Geometric design of streets should fully consider the types of traffic control to be provided, especially at intersections where multi-phase or actuated traffic signals are likely required. Signal progression, signal phasing (including pedestrian and cyclists phases), and traffic flow rates are important considerations in signalised intersection design.

9.3.8 Structures

The design of bridges, culverts, walls, tunnels, and other structures should be in accordance with the Abu Dhabi *Road Structures Design Manual* (29). The full width of the approach roadway, including sidewalks, should be continued across all new structures.

Road overpasses, sign structures, and pedestrian bridges should have a 6.0-m clearance (to the closest feature on the ceiling) over the entire roadway width. The designer should consider providing an additional clearance to allow for future resurfacing. For overpasses over railroads, the minimum vertical clearance is 7.5 m from the top of the rail.

For entrances and junctions near the structure, ensure sufficient sight distance is available. This may require widening the structure or move the entrance or junctions away from the structure to ensure sight distances are available

Ensure all structural elements have adequate ZOI (Zone of Intrusion) between them and any required road restraint system; refer to TR-518.

9.3.9 Intersections

A large proportion of crashes in urban areas occur at intersections. Safety improvements at intersections can be accomplished by improved traffic signal operations, channelizing improvements, improvement in traffic control device placement, providing appropriate sight distances, adding or increasing overhead lighting, and providing pedestrian refuge islands and pedestrian realms. Chapter 10 “Intersections” provides guidance on the design of intersections including left-turn warrants, turn-lane designs, dual-turning lanes, curb radii, channelization, etc. The Abu Dhabi *Traffic Signals and Electronic Warning and Systems Manual* (45), *Manual on Uniform Traffic Control Devices* (22), and *Road Lighting Manual* (46) provide guidance on traffic signals, traffic control devices, and roadway lighting, respectively.

For urban area, the most desirable goal is to design the intersection to minimise the interaction between pedestrians and vehicles. Consequently, intersections should be designed to:

- Accommodate the needs and accessibility of all modes of transport.
- Ensure a hierarchy of users:
  - Vulnerable users (pedestrians) first.
  - Least vulnerable (motor vehicles) last.
- Be as compact as possible.
- Minimise conflicts between modes sharing the same location at the same time.
- Provide good visibility, particularly between pedestrians and motorists. Trees, signs, and other street furniture should not obstruct visibility.
- Avoid extreme angles and complex junctions.
- Minimise pedestrian exposure to moving vehicles by reducing crossing distance.

9.3.10 Drainage

Urban streets and pedestrian realms must be designed to ensure positive drainage to avoid standing water. Surface runoff is gathered by a system of gutters, inlets, catch basins, and storm sewers. Inlets or catch basins with an open grate should be located in the gutter line and be spaced so that ponding of water on the pavement does not exceed tolerable limits. Place drainage inlets on the uphill side of the crosswalk and outside the crosswalk area. In addition, grates should be designed to accommodate cyclist and pedestrian traffic and easy cleaning after sand storms. For additional details, see the Abu Dhabi *Road Drainage Manual* (26).

9.4 Horizontal Alignment

9.4.1 Superelevation Methodology

For urban facilities with design speeds greater than 70 km/h, use the open roadway design criteria as discussed in Chapter 5 “Horizontal Alignment.” Because of the unique operational conditions for low-speed urban streets (i.e. ≤ 70 km/h), it is appropriate to use a modified theoretical basis for horizontal alignment criteria when compared to open-roadway conditions. Specifically, the use of AASHTO Method 2 to distribute superelevation and side friction. This Method assumes maximum design side friction is used before any superelevation is introduced. The practical benefit is that most horizontal curves can be designed with little or no superelevation on low-speed urban streets when compared to the criteria for high-speed urban streets and open-roadway conditions. See the AASHTO publication *A Policy on Geometric Design of Highways and Streets* (1) for a further guidance on development of Method 2.

9.4.2 General Superelevation Considerations

For low-speed urban streets, the operational conditions and physical constraints are significantly different from those on rural roads and high-speed urban streets. The following lists some of the characteristics of low-speed urban streets that often complicate superelevation development:

1. <u>Roadside Development/Intersections/Driveways</u>. Built-up roadside development is common adjacent to low-speed urban streets. Matching superelevated curves with many driveways, intersections, etc., creates considerable complications. For example, this may require reconstructing the profile on side roads, and re-grading parking lots, frontage lane, pedestrian realms, etc., to compensate for the higher elevation on the high side of the superelevated curve.

2. <u>Non-Uniform Travel Speeds</u>. On low-speed urban streets, travel speeds are often non-uniform because of frequent signalisation, stop signs, vehicular conflicts, etc. It is undesirable for traffic to stop on a superelevated curve.

3. <u>Limited Right-of-Way</u>. Superelevated curves often result in more right-of-way impacts than would otherwise be necessary. Right-of-way is often restricted along low-speed urban streets.

4. <u>Wide Pavement Areas</u>. Many low-speed urban streets have wide pavement areas because of the number of traffic lanes, the use of a median, and/or the presence of parking lanes. In general, the wider the pavement area, the development of superelevation becomes more complicated.

5. Surface Drainage. Proper cross slope drainage on low-speed urban streets can be difficult even on sections with a normal crown.

9.4.3 Maximum Superelevation Rate

On low-speed urban city roads (i.e. ≤ 70 km/h), use an e_{max} of 4.0%.

9.4.4 Minimum Radii

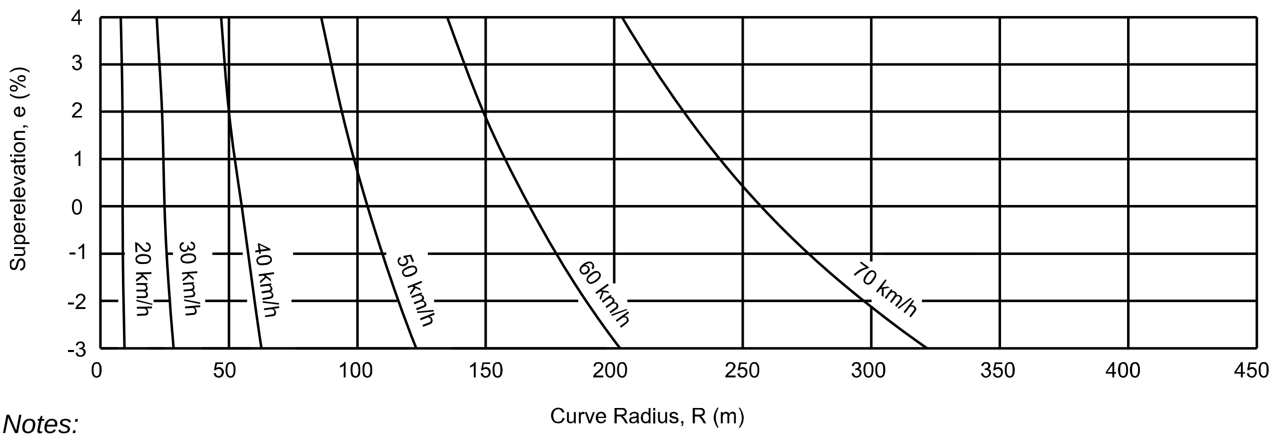
Table 9.9 presents the minimum radii for low-speed urban streets. These values should only be used where highly restricted right-of-way conditions exist.

Table 9-9: Minimum Radii on Low-Speed Urban Streets

V = Design Speed (km/h)	e_{max}	f_{max}	$e_{max} + f_{max}$	Calculated Radius (R_{min}) (m)	Radii Rounded for Design (m)
30	4.0%	0.28	0.32	22.1	22
40	4.0%	0.23	0.27	46.7	47
50	4.0%	0.19	0.23	85.6	86
60	4.0%	0.17	0.21	135.0	135
70	4.0%	0.15	0.19	203.1	203

Figure 9-9 provides a visual means to select a curve and pavement cross slope with a higher factor of safety than normally would be used for a certain design speed (i.e. providing a curve and/or superelevation rate with a higher theoretical design speed). This is a useful function provided by the format of the graph for designing horizontal curves at the fringes of urbanised areas. When motorists first enter the edge of an urbanised area, they normally take a few seconds to slow down to the posted urban speed limit, and as a result, it is desirable to provide a higher factor of safety for this speed transition area.

Figure 9-9: Superelevation Rates (Low-Speed Urban Streets)



Notes:

① *The Figure provides a range of design speeds and superelevation rates that apply to a selected curve radius.*

② *AASHTO Method 2 is used to distribute superelevation and side-friction for low-speed urban road conditions. Therefore, the basic point-mass equation applies:*

$$R = \frac{V^2}{127\,(e + f_{max})}$$

where: R = *curve radius, m*
V = *design speed, km/h*
e = *assumed superelevation rate, decimal*
f_{max} = *assumed maximum side-friction factor for selected design speed, decimal*

9.4.5 Maximum Deflection without Curve

It may be appropriate to omit a horizontal curve where very small deflection angles are present. As a guide, the designer may retain deflection angles of about 1 degree or less on low-speed urban streets. For these angles, the absence of a horizontal curve should not affect aesthetics.

9.4.6 Transition Lengths

The superelevation transition length is the distance required to transition the travelled way from a normal crown section to the full design superelevated section. The superelevation transition length is the sum of the tangent runout distance and superelevation runoff length. See Section 5.3.2. The following applies to low-speed urban streets:

1. Calculation. Section 5.3.2 presents the methodology for calculating the superelevation runoff and tangent runout for open-roadway conditions. This methodology also applies to superelevation transition lengths on low-speed urban streets. The superelevation runoff lengths (L_r) in Table 5-9 are also applicable for single carriageways, assuming the axis of rotation is about the roadway centreline. See Section 5.3.2 for guidelines on determining modifications to the superelevation transition distance where the width of rotation is more than one travel lane or where the axis of rotation is located at the edge of roadway for a two-lane road.

2. Portion of Superelevation Runoff Prior to Curve. Typically, 67% of the superelevation runoff length will be placed on tangent and 33% on curve. Exceptions to this practice may be necessary to meet field conditions. Generally, the accepted range is 60% – 80% on tangent and 40% – 20% on curve.

9.4.7 Axis of Rotation

On low-speed urban streets, the axis of rotation for horizontal curves is as follows:

1. Two-Lane Facilities. The axis of rotation is typically about one of the edges of the road.

2. Multilane Facilities. The axis of rotation is typically about the two median edges.

Low-speed urban streets may also present special problems because of the presence of turning lanes at intersections, intersections with major crossroads, drainage, etc. For these reasons, the axis of rotation is determined on a case-by-case basis.

9.4.8 Typical Designs

Section 3 provides illustrations for a typical superelevation transition design for an urban street with a raised median and for two-lane roadways, which are also applicable to two-lane streets.

9.5 Tables of Design Criteria

Table 9-10 through Table 9-14 provide the design criteria for city, town, commercial, residential, and industrial boulevards, respectively. Table 9-15 through Table 9-19 provide the design criteria for city, town, commercial, and industrial avenues, respectively. Table 9-20 through Table 9-24 provide the design criteria for city, town, commercial, and industrial streets, respectively. Table 9-25 through Table 9-29 provide the design criteria for city, town, commercial, and industrial access lanes, respectively. For the design criteria of other low-volume streets and where there is too much or too little right-of-way available, see the *Abu Dhabi Urban Street Design Manual* (4). Where it has been decided that the cross section will include a depressed median and/or outside shoulders without curbs, use the criteria presented in Chapter 8 “Rural Roads.” For all the above tables, it is important to note that for streets located in level terrain, the maximum grade is generally considered to be less than 3%. The maximum grade for streets located in level terrain that is shown in all tables mentioned above are intended for special situations as approved by the relevant authority.

The designer should anticipate that some of the cross section elements included in the tables are not automatically warranted in the project design. The values in the tables only apply <u>after</u> the decision has been made to include the element in the roadway cross section. For example, the values for a median only apply after the decision has been made to include a median in the design.

For shared cycle and pedestrian paths, a minimum width of 3 m is to be used with signage to designated shared use areas. The maximum width of frontage lanes for boulevards and avenues is to be 4.5m where diagonal parking is included instead of parallel parking.

Table 9-10: Geometric Design Criteria — City Boulevard

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	Des: 80 km/h Min: 60 km/h Max 80 km/h
	Access Control		9.2.3	Restricted Access
	Level of Service	Pedestrians	9.3.2	Des: A Min: D
		Transit		Des: B Min: D
		Cycles		Des: B Min: D
		Motorised Vehicles		Des: C Min: D
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m
		Number of Lanes (each direction)	9.3.6	3 + 3
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**
		Parking Lane	9.3.6	n/a
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
		Auxiliary Lanes	9.3.6	1.5 to 2%
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m
		Parking **(3)**	9.3.6	2.7 m
	Cycles	Lane	9.3.6	n/a
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m
	Pedestrian Realm **(4)**		9.3.6	Min.: 4.7 m Max.: 11.0 m
	Right-of-Way Width		9.3.6	**(5)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Boulevard under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Boulevard over Railroad)		6.4	7.5 m

Alignment			Manual Section	Design Speed 40 km/h	50 km/h	60 km/h	70 km/h
	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m	105 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m	225 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m	127 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m	203 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11	17
		Sag	6.3	9	13	18	23
	Maximum Grade	Level	6.2	8%	8%	7%	6%
		Rolling		9%	9%	8%	8%
		Mountainous		11%	11%	10%	9%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%			

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, DMTmunicipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-11: Geometric Design Criteria — Town Boulevard

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	Des: 80 km/h Min: 60 km/h
	Access Control		9.2.3	Restricted Access
	Level of Service	Pedestrians	9.3.2	Des: A Min: D
		Transit		Des: B Min: D
		Cycles		Des: B Min: D
		Motorised Vehicles		Des: C Min: D
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m
		Number of Lanes (each direction)	9.3.6	3 + 3
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**
		Parking Lane	9.3.6	n/a
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
		Auxiliary Lanes	9.3.6	1.5 to 2%
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m
		Parking **(3)**	9.3.6	2.7 m
	Cycles	Lane	9.3.6	n/a
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m
	Pedestrian Realm **(4)**		9.3.6	Min.: 4.3 m Max.: 10.0 m
	Right-of-Way Width		9.3.6	**(5)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Boulevard under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Boulevard over Railroad)		6.4	7.5 m

Alignment			Manual Section	Design Speed 40 km/h	50 km/h	60 km/h	70 km/h
Alignment	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m	105 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m	225 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m	127 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m	203 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11	17
		Sag	6.3	9	13	18	23
	Maximum Grade	Level	6.2	8%	8%	7%	6%
		Rolling		9%	9%	8%	8%
		Mountainous		11%	11%	10%	9%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%			

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-12: Geometric Design Criteria — Commercial Boulevard

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	Des: 80 km/h Min: 60 km/h
	Access Control		9.2.3	Restricted Access
	Level of Service	Pedestrians	9.3.2	Des: A Min: D
		Transit		Des: B Min: D
		Cycles		Des: B Min: D
		Motorised Vehicles		Des: C Min: D
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m
		Number of Lanes (each direction)	9.3.6	3 + 3
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**
		Parking Lane	9.3.6	n/a
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
		Auxiliary Lanes	9.3.6	1.5 to 2%
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m
		Parking **(3)**	9.3.6	2.7 m
	Cycles	Lane	9.3.6	n/a
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m
	Pedestrian Realm **(4)**		9.3.6	Min.: 4.3 m Max.: 10.0 m
	Right-of-Way Width		9.3.6	**(5)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Boulevard under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Boulevard over Railroad)		6.4	7.5 m

Alignment			Manual Section	Design Speed			
				40 km/h	**50 km/h**	**60 km/h**	**70 km/h**
	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m	105 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m	225 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m	127 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m	203 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11	17
		Sag	6.3	9	13	18	23
	Maximum Grade	Level	6.2	8%	8%	7%	6%
		Rolling		9%	9%	8%	8%
		Mountainous		11%	11%	10%	9%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%			

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-13: Geometric Design Criteria — Residential/Emirati Neighbourhood Boulevard

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	Des: 80 km/h Min: 60 km/h
	Access Control		9.2.3	Restricted Access
	Level of Service	Pedestrians	9.3.2	Des: A Min: D
		Transit		Des: B Min: D
		Cycles		Des: B Min: D
		Motorised Vehicles		Des: C Min: D
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m
		Number of Lanes (each direction)	9.3.6	3 + 3
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**
		Parking Lane	9.3.6	n/a
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
		Auxiliary Lanes	9.3.6	1.5 to 2%
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m
		Parking **(3)**	9.3.6	2.7 m
	Cycles	Lane	9.3.6	n/a
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m
	Pedestrian Realm **(4)**		9.3.6	Min.: 3.7 m Max.: 8.5 m
	Right-of-Way Width		9.3.6	**(5)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Boulevard under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Boulevard over Railroad)		6.4	7.5 m

Alignment			Manual Section	Design Speed 40 km/h	50 km/h	60 km/h	70 km/h
	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m	105 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m	225 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m	127 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m	203 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11	17
		Sag	6.3	9	13	18	23
	Maximum Grade	Level	6.2	8%	8%	7%	6%
		Rolling		9%	9%	8%	8%
		Mountainous		11%	11%	10%	9%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%			

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-14: Geometric Design Criteria — Industrial Boulevard

Design Element			Manual Section	Design Criteria			
Design Controls	Design Year		2.7	20 Years			
	Design Speed		9.3.1	Des: 80 km/h Min: 60 km/h			
	Access Control		9.2.3	Restricted Access			
	Level of Service	Pedestrians	9.3.2	Des: B Min: D			
		Transit		Des: B Min: D			
		Cycles		Des: B Min: D			
		Motorised Vehicles		Des: C Min: D			
Cross Section Elements	Travelled Way	Lane Width	9.3.6	Min.: 3.3 m Max.: 3.7 m			
		Number of Lanes (each direction)	9.3.6	3 + 3			
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.7 m **(1)**			
		Parking Lane	9.3.6	n/a			
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.3 m Max.: 3.7 m			
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%			
		Auxiliary Lanes	9.3.6	1.5 to 2%			
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m			
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m			
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m			
		Parking **(3)**	9.3.6	2.7 m			
	Cycles	Lane	9.3.6	n/a			
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m			
	Pedestrian Realm **(4)**		9.3.6	Min.: 3.7 m Max.: 8.5 m			
	Right-of-Way Width		9.3.6	**(5)**			
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width			
	Vertical Clearance (Boulevard under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m			
	Vertical Clearance (Boulevard over Railroad)		6.4	7.5 m			
Alignment				**Design Speed**			
				40 km/h	**50 km/h**	**60 km/h**	**70 km/h**
	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m	105 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m	225 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m	127 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m	203 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11	17
		Sag	6.3	9	13	18	23
	Maximum Grade	Level	6.2	8%	8%	7%	6%
		Rolling		9%	9%	8%	8%
		Mountainous		11%	11%	10%	9%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%			

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4)for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-15: Geometric Design Criteria — City Avenue

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	Des: 60 km/h		
	Access Control		9.2.3	Restricted Access		
	Level of Service	Pedestrians	9.3.2	Des: A Min: D		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m		
		Number of Lanes (each direction)	9.3.6	2 + 2		
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**		
		Parking Lane	9.3.6	2.5 m		
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
		Auxiliary Lanes	9.3.6	1.5 to 2%		
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m		
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m		
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m		
		Parking **(3)**	9.3.6	2.7 m		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0m		
	Pedestrian Realm **(4)**		9.3.6	Min.: 4.1 m Max.: 10.1 m		
	Right-of-Way Width		9.3.6	**(5)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Avenue under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Avenue over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				40 km/h	**50 km/h**	**60 km/h**
	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11
		Sag	6.3	9	13	18
	Maximum Grade	Level	6.2	8%	8%	7%
		Rolling		9%	9%	8%
		Mountainous		11%	11%	10%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-16: Geometric Design Criteria — Town Avenue

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
Design Controls	Design Speed		9.3.1	Des: 60 km/h		
Design Controls	Access Control		9.2.3	Restricted Access		
Design Controls	Level of Service	Pedestrians	9.3.2	Des: A Min: D		
Design Controls	Level of Service	Transit	9.3.2	Des: B Min: D		
Design Controls	Level of Service	Cycles	9.3.2	Des: B Min: D		
Design Controls	Level of Service	Motorised Vehicles	9.3.2	Des: C Min: D		
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m		
Cross Section Elements	Travelled Way	Number of Lanes (each direction)	9.3.6	2 + 2		
Cross Section Elements	Travelled Way	Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**		
Cross Section Elements	Travelled Way	Parking Lane	9.3.6	2.5 m		
Cross Section Elements	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m		
Cross Section Elements	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
Cross Section Elements	Cross Slope	Auxiliary Lanes	9.3.6	1.5 to 2%		
Cross Section Elements	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m		
Cross Section Elements	Median **(2)**	Side	9.3.6	Min.: 0.5 m Max.: 4.0 m		
Cross Section Elements	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.:4.5 m		
Cross Section Elements	Frontage Lane	Parking **(3)**	9.3.6	2.7 m		
Cross Section Elements	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
Cross Section Elements	Cycles	Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
Cross Section Elements	Pedestrian Realm **(4)**		9.3.6	Min.: 3.7 m Max.: 8.9 m		
Cross Section Elements	Right-of-Way Width		9.3.6	**(5)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
Bridges	Vertical Clearance (Avenue under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
Bridges	Vertical Clearance (Avenue over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
Alignment				**40 km/h**	**50 km/h**	**60 km/h**
Alignment	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m
Alignment	Decision Sight Distance		4.4	n/a m	155 m	195 m
Alignment	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m
Alignment	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m
Alignment	Vertical Curvature (K-values)	Crest	6.3	4	7	11
Alignment	Vertical Curvature (K-values)	Sag	6.3	9	13	18
Alignment	Maximum Grade	Level	6.2	8%	8%	7%
Alignment	Maximum Grade	Rolling	6.2	9%	9%	8%
Alignment	Maximum Grade	Mountainous	6.2	11%	11%	10%
Alignment	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-17: Geometric Design Criteria — Commercial Avenue

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	Des: 60 km/h		
	Access Control		9.2.3	Restricted Access		
	Level of Service	Pedestrians	9.3.2	Des: A Min: D		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m		
		Number of Lanes (each direction)	9.3.6	2 + 2		
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**		
		Parking Lane	9.3.6	2.5 m		
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
		Auxiliary Lanes	9.3.6	1.5 to 2%		
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 6.0 m		
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m		
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m		
		Parking **(3)**	9.3.6	2.7 m		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
	Pedestrian Realm **(4)**		9.3.6	Min.: 3.7 m Max.: 8.9 m		
	Right-of-Way Width		9.3.6	**(5)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Avenue under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Avenue over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				40 km/h	**50 km/h**	**60 km/h**
	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11
		Sag	6.3	9	13	18
	Maximum Grade	Level	6.2	8%	8%	7%
		Rolling		9%	9%	8%
		Mountainous		11%	11%	10%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the Abu Dhabi Urban Street Design Manual (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The m ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT Abu Dhabi Urban Street Design Manual for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-18: Geometric Design Criteria — Residential/Emirati Neighbourhood Avenue

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	Des: 60 km/h
	Access Control		9.2.3	Restricted Access
	Level of Service	Pedestrians	9.3.2	Des: A Min: D
		Transit		Des: B Min: D
		Cycles		Des: B Min: D
		Motorised Vehicles		Des: C Min: D
Cross Section Elements	Travelled Way	Lane Width	9.3.6	3.3 m
		Number of Lanes (each direction)	9.3.6	2 + 2
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**
		Parking Lane	9.3.6	2.5 m
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
		Auxiliary Lanes	9.3.6	1.5 to 2%
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 5.0 m
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m
		Parking **(3)**	9.3.6	2.7 m
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m
	Pedestrian Realm **(4)**		9.3.6	Min.: 3.5 m Max.: 8.0 m
	Right-of-Way Width		9.3.6	**(5)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Avenue under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Avenue over Railroad)		6.4	7.5 m

Alignment			Manual Section	Design Speed 40 km/h	Design Speed 50 km/h	Design Speed 60 km/h
Alignment	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11
		Sag	6.3	9	13	18
	Maximum Grade	Level	6.2	8%	8%	7%
		Rolling		9%	9%	8%
		Mountainous		11%	11%	10%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-19: Geometric Design Criteria — Industrial Avenue

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	Des: 60 km/h		
	Access Control		9.2.3	Restricted Access		
	Level of Service	Pedestrians	9.3.2	Des: B Min: D		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Lane Width	9.3.6	Min.: 3.3 m Max.: 3.7 m		
		Number of Lanes (each direction)	9.3.6	2 + 2		
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.7 m **(1)**		
		Parking Lane	9.3.6	Min.: 3.3 m Max.: 3.7 m		
	Auxiliary Lanes	Lane Width	9.3.6	Min.: 3.3 m Max.: 3.7 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
		Auxiliary Lanes	9.3.6	1.5 to 2%		
	Median **(2)**	Centre	9.3.6	Min.: 2.0 m Max.: 5.0 m		
		Side	9.3.6	Min.: 0.5 m Max.: 4.0 m		
	Frontage Lane	Travel Lane/Cycle	9.3.6	Min.: 4.0 m Max.: 4.5 m		
		Parking **(3)**	9.3.6	2.7 m		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
	Pedestrian Realm **(4)**		9.3.6	Min.: 3.5 m Max.: 8.3 m		
	Right-of-Way Width		9.3.6	**(5)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Avenue under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Avenue over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				40 km/h	**50 km/h**	**60 km/h**
	Stopping Sight Distance (Level)		4.3	50 m	65 m	85 m
	Decision Sight Distance		4.4	n/a m	155 m	195 m
	Intersection Sight Distance **(6)**		4.6	73 m	91 m	109 m
	Minimum Radii (e_{max} = 0.04)		9.4	47 m	86 m	135 m
	Vertical Curvature (K-values)	Crest	6.3	4	7	11
		Sag	6.3	9	13	18
	Maximum Grade	Level	6.2	8%	8%	7%
		Rolling		9%	9%	8%
		Mountainous		11%	11%	10%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Median. Use 2.0 m with a pedestrian refuge or 3.0 m where there is high pedestrian activity.
3. Frontage Lane Parking. Width only applies to a parallel parking lane next to the travelled way.
4. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
5. Right-of-Way Widths. The ROW width will be the sum of the travel lane widths, frontage lane, median widths, frontage road, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMTDMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.
6. Intersection Sight Distance. Assumes passenger vehicle turning right. ISD for left turning vehicle will be determined based on the number of lanes crossed and width of median; see Section 4.6 for guidance.

Table 9-20: Geometric Design Criteria — City Street

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	40 km/h		
	Access Control		9.2.3	Access		
	Level of Service	Pedestrians	9.3.2	Des: A Min: D		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.0 m Max.: 3.5 m **(1)**		
		Number of Lanes (each direction)	9.3.6	1 + 1		
		Parking Lane	9.3.6	4.5 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
	Pedestrian Realm **(2)**		9.3.6	Min.: 4.1 m Max.: 8.9 m		
	Right-of-Way Width		9.3.6	**(3)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Street under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Street over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				30 km/h	**40 km/h**	**50 km/h**
	Stopping Sight Distance (Level)		4.3	35 m	50 m	65 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m	84 m	105 m
		To the Left (Turning Right)		55 m	73 m	91 m
	Minimum Radii (e_{max} = 0.04)		9.4	22 m	47 m	86 m
	Vertical Curvature (K-values)	Crest	6.3	2	4	7
		Sag	6.3	6	9	13
	Maximum Grade	Level	6.2	9%	9%	9%
		Rolling		12%	12%	11%
		Mountainous		14%	13%	12%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
3. Right-of-Way Widths. The ROW width will be the sum of the travelled way width, cycle lane, auxiliary lane, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-21: Geometric Design Criteria — Town Street

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	40 km/h		
	Access Control		9.2.3	Access		
	Level of Service	Pedestrians	9.3.2	Des: A Min: D		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.0 m Max.: 3.5 m **(1)**		
		Number of Lanes (each direction)	9.3.6	1 + 1		
		Parking Lane	9.3.6	4.5 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
	Pedestrian Realm **(2)**		9.3.6	Min.: 3.7 m Max.: 7.9 m		
	Right-of-Way Width		9.3.6	**(3)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Street under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Street over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				30 km/h	**40 km/h**	**50 km/h**
	Stopping Sight Distance (Level)		4.3	35 m	50 m	65 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m	84 m	105 m
		To the Left (Turning Right)		55 m	73 m	91 m
	Minimum Radii (e_{max} = 0.04)		9.4	22 m	47 m	86 m
	Vertical Curvature (K-values)	Crest	6.3	2	4	7
		Sag	6.3	6	9	13
	Maximum Grade	Level	6.2	9%	9%	9%
		Rolling		12%	12%	11%
		Mountainous		14%	13%	12%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
3. Right-of-Way Widths. The ROW width will be the sum of the travelled way width, cycle lane, auxiliary lane, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-22: Geometric Design Criteria — Commercial Street

	Design Element		Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	40 km/h		
	Access Control		9.2.3	Access		
	Level of Service	Pedestrians	9.3.2	Des: A Min: D		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.0 m Max.: 3.5 m **(1)**		
		Number of Lanes (each direction)	9.3.6	1 + 1		
		Parking Lane	9.3.6	4.5 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
	Pedestrian Realm **(2)**		9.3.6	Min.: 3.7 m Max.: 7.9 m		
	Right-of-Way Width		9.3.6	**(3)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Street under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Street over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				30 km/h	**40 km/h**	**50 km/h**
	Stopping Sight Distance (Level)		4.3	35 m	50 m	65 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m	84 m	105 m
		To the Left (Turning Right)		55 m	73 m	91 m
	Minimum Radii (e_{max} = 0.04)		9.4	22 m	47 m	86 m
	Vertical Curvature (K-values)	Crest	6.3	2	4	7
		Sag	6.3	6	9	13
	Maximum Grade	Level	6.2	9%	9%	9%
		Rolling		12%	12%	11%
		Mountainous		14%	13%	12%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
3. Right-of-Way Widths. The ROW width will be the sum of the travelled way width, cycle lane, auxiliary lane, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-23: Geometric Design Criteria — Residential/Emirati Neighbourhood Street

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	40 km/h		
	Access Control		9.2.3	Access		
	Level of Service	Pedestrians	9.3.2	Des: A Min: D		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.0 m Max.: 3.5 m **(1)**		
		Number of Lanes (each direction)	9.3.6	1 + 1		
		Parking Lane	9.3.6	4.5 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
	Pedestrian Realm **(2)**		9.3.6	Min.: 2.0 m Max.: 5.4 m		
	Right-of-Way Width		9.3.6	**(3)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Street under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Street over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				30 km/h	**40 km/h**	**50 km/h**
	Stopping Sight Distance (Level)		4.3	35 m	50 m	65 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m	84 m	105 m
		To the Left (Turning Right)		55 m	73 m	91 m
	Minimum Radii (e_{max} = 0.04)		9.4	22 m	47 m	86 m
	Vertical Curvature (K-values)	Crest	6.3	2	4	7
		Sag	6.3	6	9	13
	Maximum Grade	Level	6.2	9%	9%	9%
		Rolling		12%	12%	11%
		Mountainous		14%	13%	12%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
3. Right-of-Way Widths. The ROW width will be the sum of the travelled way width, cycle lane, auxiliary lane, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-24: Geometric Design Criteria — Industrial Street

Design Element			Manual Section	Design Criteria		
Design Controls	Design Year		2.7	20 Years		
	Design Speed		9.3.1	40 km/h		
	Access Control		9.2.3	Access		
	Level of Service	Pedestrians	9.3.2	Des: B Min: C		
		Transit		Des: B Min: D		
		Cycles		Des: B Min: D		
		Motorised Vehicles		Des: C Min: D		
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.3 m Max.: 3.7 m		
		Number of Lanes (each direction)	9.3.6	1 + 1		
		Curb/Transit Lane	9.3.6	Min.: 3.3 m Max.: 3.5 m **(1)**		
		Parking Lane	9.3.6	4.5 m		
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%		
	Cycles	Lane	9.3.6	Min.: 1.5 m Max.: 2.5 m		
		Track	9.3.6	Min.: 1.5 m Max.: 3.0 m		
	Pedestrian Realm **(2)**		9.3.6	Min.: 3.5 m Max.: 7.0 m		
	Right-of-Way Width		9.3.6	**(3)**		
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width		
	Vertical Clearance (Street under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m		
	Vertical Clearance (Street over Railroad)		6.4	7.5 m		
Alignment				**Design Speed**		
				30 km/h	**40 km/h**	**50 km/h**
	Stopping Sight Distance (Level)		4.3	35 m	50 m	65 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m	84 m	105 m
		To the Left (Turning Right)		55 m	73 m	91 m
	Minimum Radii (e_{max} = 0.04)		9.4	22 m	47 m	86 m
	Vertical Curvature (K-values)	Crest	6.3	2	4	7
		Sag	6.3	6	9	13
	Maximum Grade	Level	6.2	9%	9%	9%
		Rolling		12%	12%	11%
		Mountainous		14%	13%	12%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%		

1. Curb/Transit Lane. Provide a 3.5-m curb lane where buses use the curb lane as part of the regular transit route.
2. Pedestrian Realm. Widths are for total width of pedestrian realm, excluding the cycle track. See the *Abu Dhabi Urban Street Design Manual* (4)for detailed guidance on pedestrian realms.
3. Right-of-Way Widths. The ROW width will be the sum of the travelled way width, cycle lane, auxiliary lane, and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-25: Geometric Design Criteria — City Access Lane

<table>
<tr><th colspan="4">Design Element</th><th>Manual Section</th><th>Design Criteria</th></tr>
<tr><td rowspan="7">Design Controls</td><td colspan="3">Design Year</td><td>2.7</td><td>20 Years</td></tr>
<tr><td colspan="3">Design Speed</td><td>9.3.1</td><td>30 km/h</td></tr>
<tr><td colspan="3">Access Control</td><td>9.2.3</td><td>Access</td></tr>
<tr><td rowspan="4">Level of Service</td><td colspan="2">Pedestrians</td><td rowspan="4">9.3.2</td><td>A</td></tr>
<tr><td colspan="2">Transit</td><td>n/a</td></tr>
<tr><td colspan="2">Cycles</td><td>B</td></tr>
<tr><td colspan="2">Motorised Vehicles</td><td>n/a</td></tr>
<tr><td rowspan="6">Cross Section Elements</td><td rowspan="3">Travelled Way</td><td colspan="2">Curb Lane Width</td><td>9.3.6</td><td>Min.: 3.0 m Max.: 3.3 m</td></tr>
<tr><td colspan="2">Number of Lanes (each direction)</td><td>9.3.6</td><td>1 + 1/1 shared</td></tr>
<tr><td colspan="2">Parking Lane</td><td>9.3.6</td><td>4.5 m</td></tr>
<tr><td>Cross Slope</td><td colspan="2">Travel Lane</td><td>9.3.6</td><td>1.5 to 2%</td></tr>
<tr><td colspan="3">Pedestrian Realm (1)</td><td>9.3.6</td><td>Min.: 2.0 m Max.: 4.0 m</td></tr>
<tr><td colspan="3">Right-of-Way Width</td><td>9.3.6</td><td>(2)</td></tr>
<tr><td rowspan="3">Bridges</td><td colspan="3">Clear Roadway Width</td><td>9.3.7</td><td>Approach Roadway Width</td></tr>
<tr><td colspan="3">Vertical Clearance (Access Lanes under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)</td><td>6.4</td><td>6.0 m</td></tr>
<tr><td colspan="3">Vertical Clearance (Access Lane over Railroad)</td><td>6.4</td><td>7.5 m</td></tr>
<tr><td rowspan="14">Alignment</td><td colspan="3" rowspan="2"></td><td rowspan="2"></td><td>Design Speed</td></tr>
<tr><td>20 km/h</td></tr>
<tr><td colspan="3">Stopping Sight Distance (Level)</td><td>4.3</td><td>20 m</td></tr>
<tr><td rowspan="2">Intersection Sight Distance</td><td colspan="2">To the Right (Turning Left)</td><td rowspan="2">4.6</td><td>63 m</td></tr>
<tr><td colspan="2">To the Left (Turning Right)</td><td>55 m</td></tr>
<tr><td colspan="3">Minimum Radii (e_{max} = 0.04)</td><td>9.4</td><td>8 m</td></tr>
<tr><td colspan="2" rowspan="2">Vertical Curvature (K-values)</td><td>Crest</td><td>6.3</td><td>1</td></tr>
<tr><td>Sag</td><td>6.3</td><td>3</td></tr>
<tr><td colspan="2" rowspan="3">Maximum Grade</td><td>Level</td><td rowspan="3">6.2</td><td>9%</td></tr>
<tr><td>Rolling</td><td>12%</td></tr>
<tr><td>Mountainous</td><td>15%</td></tr>
<tr><td colspan="3">Minimum Grade</td><td>9.3.5</td><td>Desirable: 0.5%
Minimum: 0.3%</td></tr>
</table>

1. Pedestrian Realm. Widths are for total width of pedestrian realm. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
2. Right-of-Way Widths. The ROW width will be the sum of the travelled way width and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-26: Geometric Design Criteria — Town Access Lane

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	30 km/h
	Access Control		9.2.3	Access
	Level of Service	Pedestrians	9.3.2	A
		Transit		n/a
		Cycles		B
		Motorised Vehicles		n/a
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
		Number of Lanes (each direction)	9.3.6	1 + 1/1 shared
		Parking Lane	9.3.6	4.5 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
	Pedestrian Realm **(1)**		9.3.6	Min.: 2.0 m Max.: 4.0 m
	Right-of-Way Width		9.3.6	**(2)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Access Lanes under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Access Lane over Railroad)		6.4	7.5 m
Alignment				**Design Speed**
				20 km/h
	Stopping Sight Distance (Level)		4.3	20 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m
		To the Left (Turning Right)		55 m
	Minimum Radii (e_{max} = 0.04)		9.4	8 m
	Vertical Curvature (K-values)	Crest	6.3	1
		Sag	6.3	3
	Maximum Grade	Level	6.2	9%
		Rolling		12%
		Mountainous		15%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%

1. Pedestrian Realm. Widths are for total width of pedestrian realm. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
2. Right-of-Way Widths. The ROW width will be the sum of the travelled way width and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-27: Geometric Design Criteria — Commercial Access Lane

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	30 km/h
	Access Control		9.2.3	Access
	Level of Service	Pedestrians	9.3.2	A
		Transit		n/a
		Cycles		B
		Motorised Vehicles		n/a
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
		Number of Lanes (each direction)	9.3.6	1 + 1/1 shared
		Parking Lane	9.3.6	4.5 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
	Pedestrian Realm **(1)**		9.3.6	Min.: 2.0 m Max.: 4.0 m
	Right-of-Way Width		9.3.6	**(2)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Access Lanes under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Access Lane over Railroad)		6.4	7.5 m
Alignment				**Design Speed**
				20 km/h
	Stopping Sight Distance (Level)		4.3	20 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m
		To the Left (Turning Right)		55 m
	Minimum Radii (e_{max} = 0.04)		9.4	8 m
	Vertical Curvature (K-values)	Crest	6.3	1
		Sag	6.3	3
	Maximum Grade	Level	6.2	9%
		Rolling		12%
		Mountainous		15%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%

1. Pedestrian Realm. Widths are for total width of pedestrian realm. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
2. Right-of-Way Widths. The ROW width will be the sum of the travelled way width and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-28: Geometric Design Criteria — Residential/Emirati Neighbourhood Access Lane

Design Element			Manual Section	Design Criteria
Design Controls	Design Year		2.7	20 Years
	Design Speed		9.3.1	30 km/h
	Access Control		9.2.3	Access
	Level of Service	Pedestrians	9.3.2	A
		Transit		n/a
		Cycles		B
		Motorised Vehicles		n/a
Cross Section Elements	Travelled Way	Curb Lane Width	9.3.6	Min.: 3.0 m Max.: 3.3 m
		Number of Lanes (each direction)	9.3.6	1 + 1/1 shared
		Parking Lane	9.3.6	4.5 m
	Cross Slope	Travel Lane	9.3.6	1.5 to 2%
	Pedestrian Realm **(1)**		9.3.6	Min.: 2.0 m Max.: 4.9 m
	Right-of-Way Width		9.3.6	**(2)**
Bridges	Clear Roadway Width		9.3.7	Approach Roadway Width
	Vertical Clearance (Access Lanes under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)		6.4	6.0 m
	Vertical Clearance (Access Lane over Railroad)		6.4	7.5 m
Alignment				**Design Speed**
				20 km/h
	Stopping Sight Distance (Level)		4.3	20 m
	Intersection Sight Distance	To the Right (Turning Left)	4.6	63 m
		To the Left (Turning Right)		55 m
	Minimum Radii (e_{max} = 0.04)		9.4	8 m
	Vertical Curvature (K-values)	Crest	6.3	1
		Sag	6.3	3
	Maximum Grade	Level	6.2	9%
		Rolling		12%
		Mountainous		15%
	Minimum Grade		9.3.5	Desirable: 0.5% Minimum: 0.3%

1. Pedestrian Realm. Widths are for total width of pedestrian realm. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
2. Right-of-Way Widths. The ROW width will be the sum of the travelled way width and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies. See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

Table 9-29: Geometric Design Criteria — Industrial Access Lane

<table>
<tr><th colspan="4">Design Element</th><th>Manual Section</th><th>Design Criteria</th></tr>
<tr><td rowspan="7">Design Controls</td><td colspan="3">Design Year</td><td>2.7</td><td>20 Years</td></tr>
<tr><td colspan="3">Design Speed</td><td>9.3.1</td><td>30 km/h</td></tr>
<tr><td colspan="3">Access Control</td><td>9.2.3</td><td>Access</td></tr>
<tr><td rowspan="4">Level of Service</td><td colspan="2">Pedestrians</td><td rowspan="4">9.3.2</td><td>B</td></tr>
<tr><td colspan="2">Transit</td><td>n/a</td></tr>
<tr><td colspan="2">Cycles</td><td>B</td></tr>
<tr><td colspan="2">Motorised Vehicles</td><td>n/a</td></tr>
<tr><td rowspan="6">Cross Section Elements</td><td rowspan="3">Travelled Way</td><td colspan="2">Curb Lane Width</td><td>9.3.6</td><td>Min.: 3.0 m Max.: 3.7 m</td></tr>
<tr><td colspan="2">Number of Lanes (each direction)</td><td>9.3.6</td><td>1 + 1/1 shared</td></tr>
<tr><td colspan="2">Parking Lane</td><td>9.3.6</td><td>4.5 m</td></tr>
<tr><td>Cross Slope</td><td colspan="2">Travel Lane</td><td>9.3.6</td><td>1.5 to 2%</td></tr>
<tr><td colspan="3">Pedestrian Realm (1)</td><td>9.3.6</td><td>Min.: 2.0 m Max.: 4.0 m</td></tr>
<tr><td colspan="3">Right-of-Way Width</td><td>9.3.6</td><td>(2)</td></tr>
<tr><td rowspan="3">Bridges</td><td colspan="3">Clear Roadway Width</td><td>9.3.7</td><td>Approach Roadway Width</td></tr>
<tr><td colspan="3">Vertical Clearance (Access Lanes under Overcrossing Roadways, Overhead Signs, Pedestrian Bridges)</td><td>6.4</td><td>6.0 m</td></tr>
<tr><td colspan="3">Vertical Clearance (Access Lane over Railroad)</td><td>6.4</td><td>7.5 m</td></tr>
<tr><td rowspan="13">Alignment</td><td colspan="3" rowspan="2"></td><td rowspan="2"></td><td>Design Speed</td></tr>
<tr><td>20 km/h</td></tr>
<tr><td colspan="3">Stopping Sight Distance (Level)</td><td>4.3</td><td>20 m</td></tr>
<tr><td rowspan="2">Intersection Sight Distance</td><td colspan="2">To the Right (Turning Left)</td><td rowspan="2">4.6</td><td>63 m</td></tr>
<tr><td colspan="2">To the Left (Turning Right)</td><td>55 m</td></tr>
<tr><td colspan="3">Minimum Radii (e_{max} = 0.04)</td><td>9.4</td><td>8 m</td></tr>
<tr><td colspan="2" rowspan="2">Vertical Curvature (K-values)</td><td>Crest</td><td>6.3</td><td>1</td></tr>
<tr><td>Sag</td><td>6.3</td><td>3</td></tr>
<tr><td colspan="2" rowspan="3">Maximum Grade</td><td>Level</td><td rowspan="3">6.2</td><td>9%</td></tr>
<tr><td>Rolling</td><td>12%</td></tr>
<tr><td>Mountainous</td><td>15%</td></tr>
<tr><td colspan="3">Minimum Grade</td><td>9.3.5</td><td>Desirable: 0.5%
Minimum: 0.3%</td></tr>
</table>

1. Pedestrian Realm. Widths are for total width of pedestrian realm. See the *Abu Dhabi Urban Street Design Manual* (4) for detailed guidance on pedestrian realms.
2. Right-of-Way Widths. The ROW width will be the sum of the travelled way width and pedestrian realm. The ROW width must comply with the guidelines and standards in coordination with all applicable agencies (e.g. DMT, municipalities). See the DMT *Abu Dhabi Urban Street Design Manual* for design priorities where there is too much or too little ROW available.

10 INTERSECTIONS

10.1 Overview

Intersections (junctions) are an important part of the roadway system. The operational efficiency, capacity, safety, and cost of the overall system are largely dependent upon the intersection design, especially in urban areas. The primary objective of intersection design is to provide for the convenience, ease, comfort, and safety of those traversing the intersection while reducing potential conflicts between vehicles, cyclists, and pedestrians. Chapter 10 provides guidance in the design of intersections including intersection types, alignment, profile, design vehicle selection, turning radii, right-turning roadways, left- and right-turn lanes, channelized islands, and median openings.

In addition to Chapter 10, Chapter 4 “Sight Distance” discusses intersection sight distances. Table 2-5 shows the relationship between the AASHTO functional classifications used in this *Manual* with the street categories used in the Abu Dhabi Urban Planning Council *Abu Dhabi Urban Street Design Manual* (4).

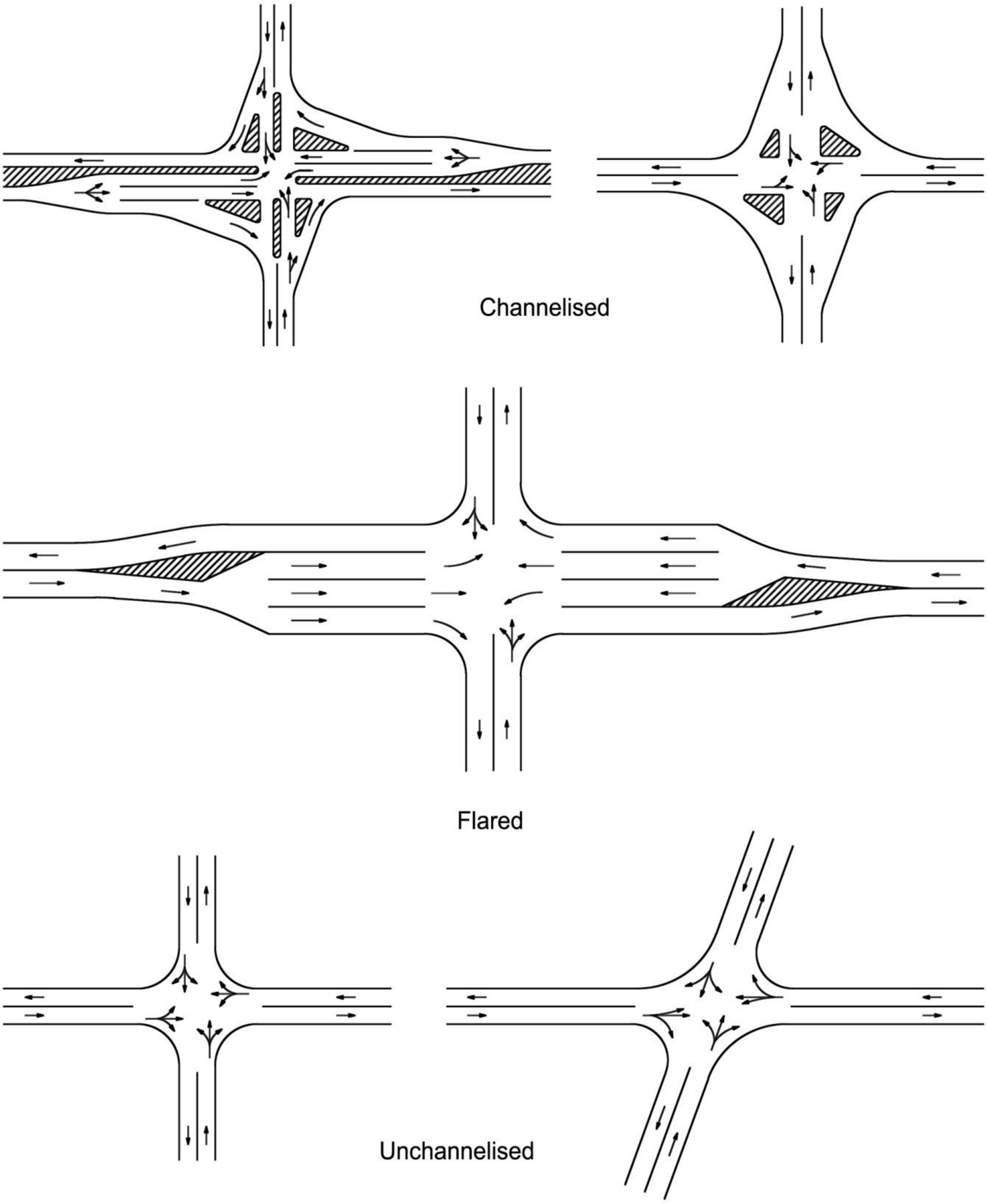


Figure 10-1: Basic Intersection Types

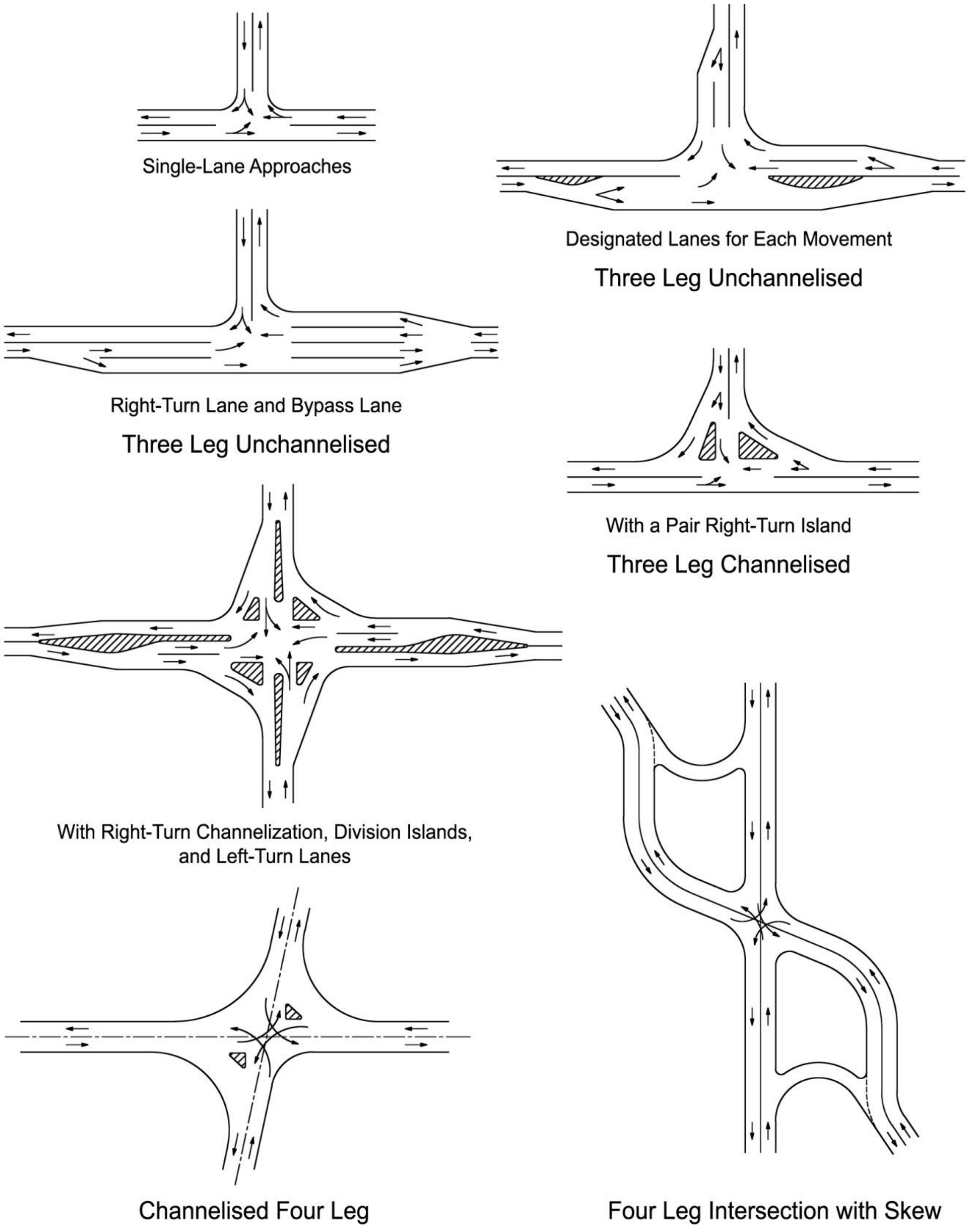


Figure 10-2: Basic Intersection Types

10.2 General Design Controls

10.2.1 General Design Considerations

In every intersection design, there are many conflicting requirements that must be balanced against each other to produce a safe and efficient design. The five basic elements that must be taken into consideration include:

1. Human Factors. These include:

    - driving habits,
    - ability to make decisions,
    - driver expectancy,
    - decision and reaction time,
    - conformance to natural paths of movement, and
    - pedestrian use and habits.

2. Transportation Considerations. These include:

    - capacity;
    - design hourly volumes (i.e., pedestrians, transit, cyclists, motor vehicles);
    - vehicular composition,
    - turning movements;
    - vehicular speeds (design and operating); and
    - safety.

3. Physical Elements. These include:

    - character and location of abutting property,
    - topography,
    - right-of-way,

    - horizontal alignment,
    - vertical alignment,
    - coordination of vertical profiles of the intersecting roads,
    - coordination of horizontal and vertical alignment for intersections on curves,
    - available sight distance,
    - intersection angle,
    - conflict area,
    - geometrics,
    - channelization,
    - traffic control devices,
    - lighting,
    - safety features,
    - transit vehicles,
    - environmental impact, and
    - drainage requirements.

4. Economic Factors. These include:

    - cost of improvements;
    - crash history;
    - effects on adjacent property (e.g. access to businesses); and
    - impact on energy.

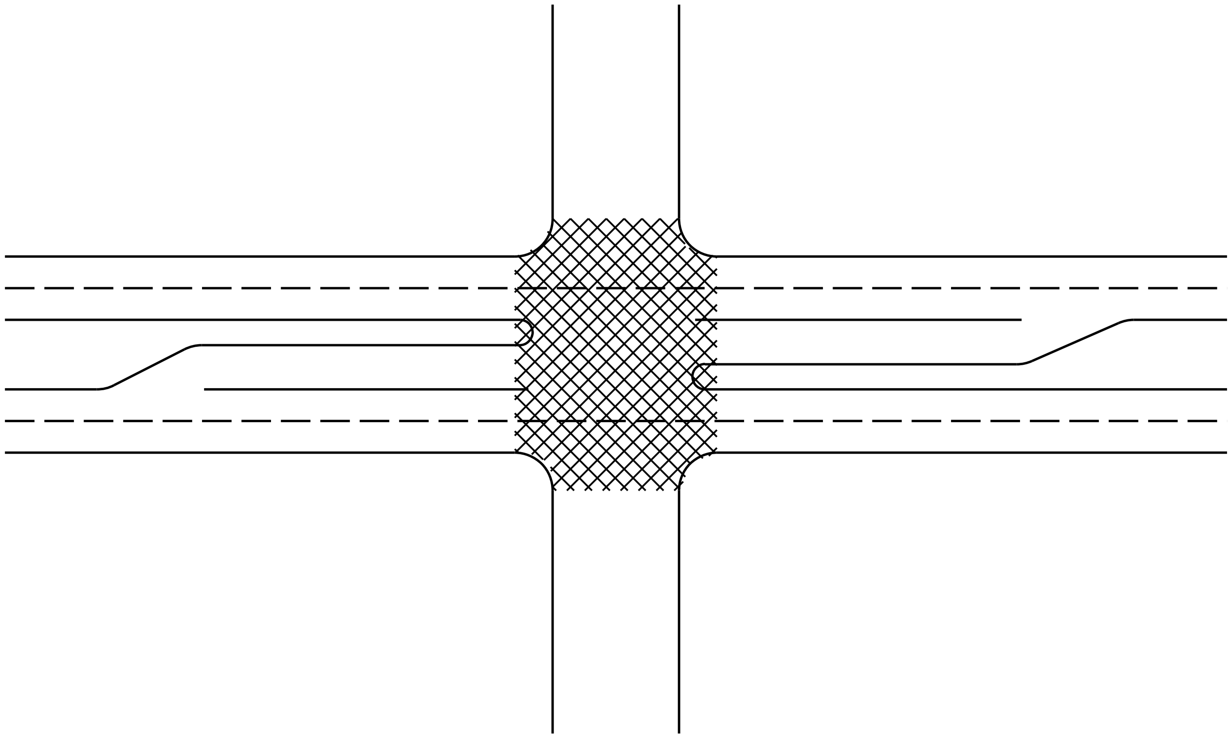
5. Functional Intersection Area. An intersection can be defined by both functional and physical areas. These are illustrated in Figure 10-3. The functional area of the intersection extends both upstream and downstream from the physical intersection area and includes any auxiliary lanes and their associated channelization.

The essence of good intersection design requires that the physical elements be designed to minimise the potential conflicts among cars, trucks, transit vehicles, cyclists, and pedestrians. In addition, human factors of the drivers and pedestrians must be taken into account while keeping costs and impacts to a minimum.

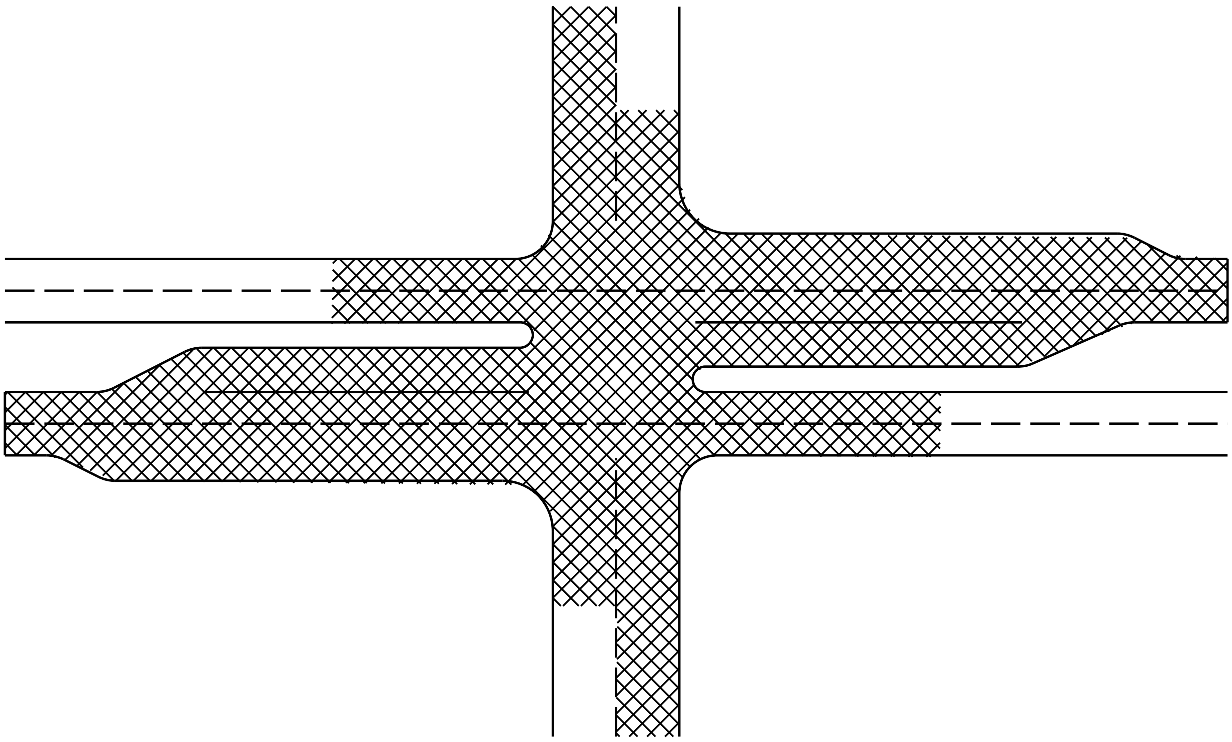
10.2.2 Intersection Components

Figure 10-4 illustrates several of the components that may be included in a typical intersection. Figure 10-4(a) illustrates the components for a typical rural intersection and Figure 10-4(b) for an urban intersection.

Figure 10-3: Physical and Functional Intersection Area



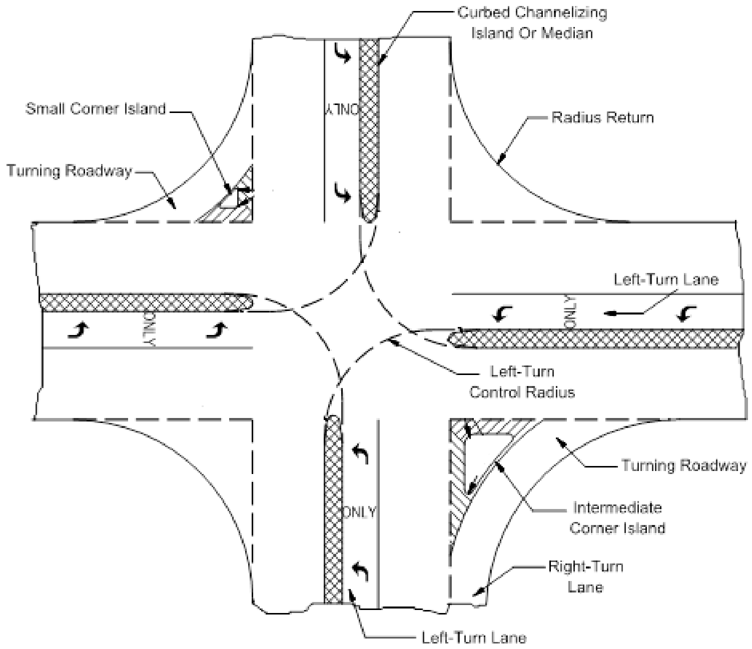
Physical Area



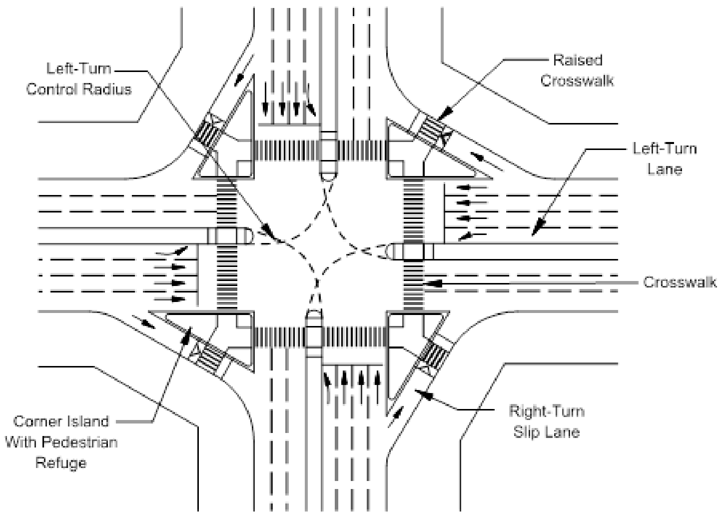
Functional Intersection Area

Source: (1)

Figure 10-4: Typical Intersection Components



(a) Rural Intersection (1)



(b) Urban Intersection (4)

Note: Provide physical separation between through and left turning movements for Lead-Lag junctions.

10.2.3 Intersection Capacity

10.2.3.1 General

Transportation volumes and characteristics of all users to be served are major factors in determining the geometric criteria for intersections/junctions. These characteristics are determinedby pedestrian demands, transit users, cyclists, and motor vehicular users. For urban streets, the designer also needs to consider all users of the intersection/junction. In selecting the appropriate level of service for urban facilities, compromises may be required between the various users. Where compromises are required, the designer should consider the design priorities as noted in Section 9.2.4. See Section 9.3.2 for guidance on determining the level of services for pedestrians, transit, and cyclists.

The *Highway Capacity Manual 2010* (HCM) (17) presents analysis techniques for comparing operation among different conditions at intersections. The HCM includes analysis techniques for intersections with a stop sign on one or two approaches, stop on all approaches, signalised intersections, and roundabout intersections. A number of analysis tools by several software developers are available that use the techniques presented in the HCM. Categories of tools include:

- Tools to analyse intersections or roadway segments and determine level of service;
- Tools to develop optimal signal phasing and timing plans for isolated intersections, arterial streets, or signal networks; and
- Tools to simulate traffic flow in an intersection, arterial street, or street network.

Intersection capacity analysis methodology uses control delay as the primary measure of driver discomfort, frustration, fuel consumption, and increased travel time. Therefore, various forms of control such as all-way stop, roundabout, and signal can be compared at an intersection using delay. For roadways on which many intersections are controlled by traffic signals, the capacity of the signalised intersections determines the capacity of the roadway.

The angle of intersection, number of through and turn lanes, lane widths, grades, channelization, etc., all affect the optimum capacity of the intersection and the intersection's level of service. Consequently, the designer must realise how small changes to the intersection design may impact the overall capacity of the intersection.

Intersection levels of service, shown in Table 10-1, are defined to represent reasonable ranges in control delay and intersection conditions.

10.2.3.2 Signalised Intersections

In the HCM approach, capacity at intersections is defined for individual lane groups and for the intersection as a whole. A lane group may be a single movement, group of movements, or entire approach and is defined by the intersection geometry and distribution of movements over the various lanes. Capacity of a lane group is calculated as the maximum rate of flow that may pass through the intersection under prevailing traffic, roadway, and signalization conditions. The rate of flow is generally measured or projected for a 15-minute period and capacity is stated in vehicles per hour. Capacity analysis of intersections involves the computation of volume-to-capacity (v/c) ratios for each lane group, from which an overall intersection v/c ratio may be derived.

Table 10-1: Level of Service Definitions for Signalised Intersections

Level of Service	Intersection Conditions
A	Very short delay; most vehicles do not stop because of short cycle lengths, favourable progression, and arrival of most vehicles during the green phase.
B	Short delay; many vehicles do not stop or stop for a short time as a result of short cycle lengths and good progression.
C	Moderate delay; many vehicles having to stop, and occasional individual cycle failures as a result of longer cycle lengths and fair progression.
D	Longer delays; many vehicles having to stop, and a noticeable number of individual cycle failures as a result of some combination of long cycle lengths, high volume to capacity ratios, and unfavourable progression.
E	Long delays; frequent cycle failures result from long cycle lengths and/or high volume to capacity ratios, which in turn results in poor progression.
F	Delays considered unacceptable to most drivers occur when the vehicle arrival rate, over extended time, is greater than the intersection's capacity.

Source: (17)

Generally, when two opposing flows are moving during a single phase, one of the lane groups will require more green time than another to process all of its volume. This would be defined as the critical lane group for the subject signal phase. The concept of a critical v/c ratio is used to evaluate the intersection as a whole, considering only the critical lane groups or those with the greatest demand for green time within each signal phase. This procedure assumes that green time has been appropriately allocated. Consequently, it is possible to have an overall intersection v/c of less than 1.00 (under capacity), but still have individual movements be over saturated within the signal cycle if the green time has not been appropriately allocated to the various approaches.

Intersection level-of-service delay criteria for signalised intersections are stated in terms of average stopped delay per vehicle for a 15-minute analysis period; see Table 10-2.

The base saturation flow rate used in the analysis model is 1,900 passenger cars per hour of green per lane. This value is adjusted for prevailing traffic conditions (e.g. lane width, left turns, right turns, heavy vehicles, grades, parking, area type, bus blockage, left-turn blockage).

If the level of service cannot be met, the designer should consider adjusting the signal timing, adding turn and/or through lanes, realigning the intersection, eliminate parking near the intersection, restrict turns during the peak hour, coordinate adjacent signals, etc.

Table 10-2: LOS Criteria for Signalised Intersections

Level of Service	Measures of Performance			
	Intersection Delay (seconds)	Intersection Capacity Utilization	Volume/ Capacity Ratio	Number of Stops Before Clearing
A	≤ 10	0.0 – 0.6	0 – 0.6	0
B	> 10 – 20	0.6 – 0.7	0.6 – 0.7	0
C	> 20 – 35	0.7 – 0.8	0.7 – 0.8	0
D	> 35 – 55	0.8 – 0.9	0.8 – 0.9	1 – 2
E	> 55 – 80	0.9 – 1.0	0.9 – 1.0	2 – 3
F	> 80	>1.0	>1.0	>3.0

Source: (18)

10.2.3.3 Unsignalised Intersections

Level of service for unsignalised intersections is based on the assumption that major street traffic is not affected by minor street movements (i.e., minor street traffic must wait for a gap in major street traffic). The capacity of the intersection to accommodate minor street movements is based on the amount of traffic on the major street and the configuration of the intersection. LOS is based on the average total delay, defined as the total elapsed time from when a vehicle stops at the end of the queue until the vehicle departs from the stop line (this time includes the time required for the vehicle to travel from the last-in-queue position to the first-in-queue position. The average total delay for any particular minor movement is a function of the service rate or capacity of the approach and the degree of saturation. Table 10-3 presents the LOS criteria for unsignalised intersections.

LOS criteria apply to each lane on a given approach and to each approach on the minor street. LOS is not calculated for major-street approaches or for the intersection as a whole.

Table 10-3: LOS Criteria for Unsignalised Intersections

Level of Service	Measures of Performance		
	Stopped Delay (seconds)	Intersection Capacity Utilizations	Number of Stops Before Clearing
A	0 – 10	0.00 – 0.55	0
B	> 10 – 15	0.55 – 0.64	0
C	> 15 – 25	0.64 – 0.73	0
D	> 25 – 35	0.73 – 0.82	1 – 2
E	> 35 – 50	0.82 – 0.91	2 – 3
F	> 50	0.91 – 1.00	> 4

Source: (18)

10.2.4 Intersection Types

10.2.4.1 General

Intersections are usually a three-leg, four-leg, or multi-leg design. Individual intersections may vary in size and shape and may be channelized. The principal design factors that affect the selection of intersection type and its design characteristics are discussed in Section 10.2.1. Selection of the intersection type will be determined on a case-by-case basis.

10.2.4.2 Three-Leg Intersections

The most common type of three-leg or Tee intersection has the normal pavement width of both roadways maintained except for the paved corner radii or where widening is needed to accommodate the selected design vehicle. This type of unchannelized intersection is generally suitable for junctions of minor or local roads and junctions of minor roads with more important roadways where the angle of intersection is not generally more than 30 degrees from perpendicular (i.e. from approximately 60 degrees to 120 degrees). In rural areas, this intersection type is usually used in conjunction with two-lane roadways carrying low traffic volumes. In urban areas, it may be satisfactory for higher volumes and for multilane roads. Where speeds and/or turning movements are high, an additional area of surfacing or flaring may be provided for manoeuvrability. Examples of a three-leg intersection are shown in Figure 10-5.

10.2.4.3 Four-Leg Intersections

The overall design principles, island arrangements, use of auxiliary lanes, and many other aspects of the previous discussion of three-leg intersection design also apply to four-leg intersections. Unchannelized four-leg intersections are suitable for intersections of minor or local roads and often suitable for intersections of minor roads with major roadways. A skewed intersection leg should not be more than 30 degrees from perpendicular (i.e. from approximately 60 degrees to 120 degrees). Approach pavements are continued through the intersection, and the corners are founded to accommodate turning vehicles.

For channelized four-leg intersections, right-turning roadways are often provided at major intersections for the more important turning movements where large vehicles are to be accommodated and at minor intersections in quadrants where the angle of turn greatly exceeds 90 degrees. Examples of four-leg intersections are shown in Figure 10-6.

10.2.4.4 Multileg Intersections

Multileg intersections, those with five or more intersections legs, should be avoided wherever practical. At locations where multileg intersections are used, it may be satisfactory to have all intersection legs intersect at a common paved area, where traffic volumes are low and stop control is used. At other than minor intersections, traffic operational efficiency can often be improved by reconfigurations that remove some conflicting movements from the major intersection. Such reconfigurations are accomplished by realigning one or more of the intersecting legs and combining some of the traffic movements at adjacent subsidiary intersections. Other options include redesigning the intersection to a roundabout or converting one or more legs to one-way operation away from the intersection. Figure 10-7 shows a multileg intersection with a roundabout.

Figure 10-5: Three-Leg Intersections

Figure 10-6: Four-Leg Intersections

Figure 10-7: Multileg Intersection

10.2.4.5 Roundabouts

A roundabout is an intersection with a central island around which traffic must travel counter clockwise and in which entering traffic must yield to circulating traffic. Not all circular intersections can be classified as roundabouts. Example roundabouts are shown in Figure 10-8.

Roundabouts can be classified into three basic categories according to size and number of lanes to facilitate discussion of specific performance and design issues:

- mini-roundabouts,
- single-lane roundabouts, and
- multilane roundabouts.

Chapter 11 “Roundabouts” provides detailed guidance on roundabouts.

10.2.5 Intersection Spacing

It is important to maintain suitable spacing between intersections in order to ensure proper functionality and capacity of street networks and individual streets. If intersections are too close to each other, they may impede traffic operations. For example, if two intersections are close together they may need to be considered as one intersection for signal phasing purposes and queuing analysis. To operate safely, each leg of the intersection may require a separate green phase; hence, this could reduce the capacity for both intersections. When introducing a new intersection, the designer must ensure that there is sufficient distance between the new and adjacent intersections so that they form distinct intersections. Also, consider the entrance for access lanes as they may impact the operation of intersections.

Figure 10-8: Roundabouts

If intersections are spaced too far apart from each other, then there may be insufficient connectivity within the overall street network and the overall capacity of the network will suffer.

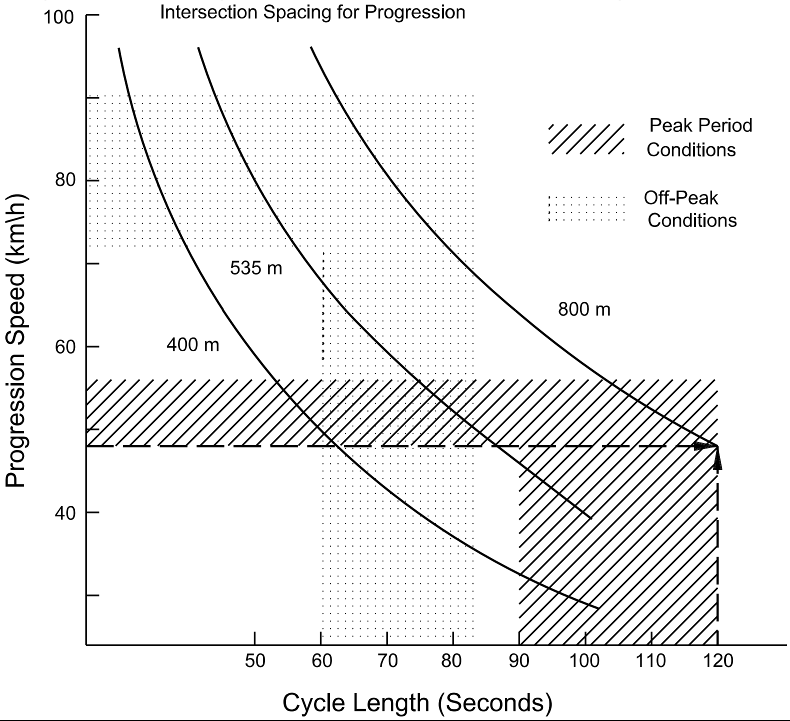
Spacing for unsignalised intersections and driveways will depend on the available stopping sight distance, intersection sight distance, traffic volumes, through lanes, turn lanes, progression speeds, access control, and local development. The actual spacing will be determined on a case-by-case basis.

The need to efficiently move high volumes of traffic, especially during peak periods, is a major consideration in the spacing of signalised intersections. It is important that the signals be synchronised to efficiently move traffic. Figure 10-9 illustrates the relationship between speed of progression, cycle length, and signalised intersection spacing.

Table 10-4 provides desirable dimensions for spacing of new urban intersections. For existing street redesigns, use the dimensions in Table 10.4 as a guide for considering additional street links and improving connectivity.

See the Abu Dhabi *Access Management Policy and Procedures Manual* (21) for further details on intersection spacing.

Figure 10-9: Signalised Intersection Spacing Guidelines



Cycle Length (sec)	Posted Speed (km/h)					
	40	50	60	70	80	90
	Intersection Spacing for Progression (2)					
60	335 m	400 m	500 m	605 m	670 m	730 m
70	390 m	470 m	590 m	705 m	760 m	800 m
80	450 m	535 m	670 m	800 m	800 m	800 m
90	495 m	605 m	755 m	800 m	800 m	800 m
120	670 m	800 m	800 m	800 m	800 m	800 m
150(1)	800 m	800 m	800 m	800 m	800 m	800 m

Notes:

1. *Represents maximum cycle length for actuated signal if all phases are used.*
2. *From a practical standpoint when considering progression, the distance between signalised intersections will usually be 800 m or less. Therefore, the values in the table have been limited to 800 m.*

Source: (47)

Table 10-4: Urban Intersection Spacing Criteria

Context		Arterial	Collector	Local Street
City	Min	400 m	200 m	100 m
	Max	750 m	375 m	175 m
Town	Min	600 m	300 m	140 m
	Max	1000 m	500 m	250 m
Commercial	Min	1000 m	400 m	125 m
	Max	1500 m	750 m	375 m
Residential	Min	1000 m	400 m	125 m
	Max	1500 m	750 m	375 m
Industrial	Min	800 m	400 m	-
	Max	1500 m	750 m	300 m

Source: (4)

10.2.6 Intersection Alignment

10.2.6.1 Angled Intersections

Roadways should intersect at right angles. Intersections at acute angles are undesirable because they:

- restrict vehicular turning movements,
- require additional pavement and channelization for large trucks,
- increase the exposure time for vehicles and pedestrians crossing the main traffic flow, and
- restrict the crossroad sight distance.

Preferably, the angle of intersection should be within 15 degrees of perpendicular. This amount of skew can often be tolerated because the impact on sight lines and turning movements is not significant. Under restricted conditions where obtaining the right-of-way to straighten the angle of intersection would be impractical, an intersection angle up to 30 degrees from perpendicular may be used. Where turning movements are significantly unbalanced, the intersections may be angled to favour the predominant movement. Intersection angles beyond these ranges may warrant more positive traffic control (e.g. all stop, traffic signals) or geometric improvements (e.g. realignment, greater intersection sight distance). Figure 10-10 shows an example of an angled intersection.

Figure 10-10: Angled Intersection

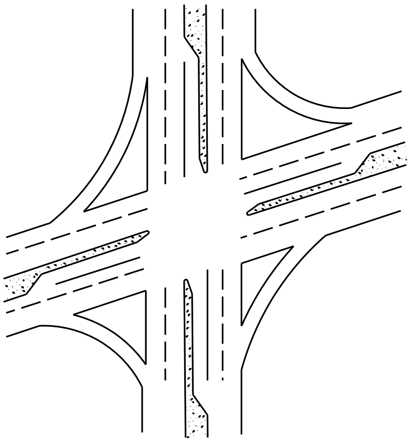
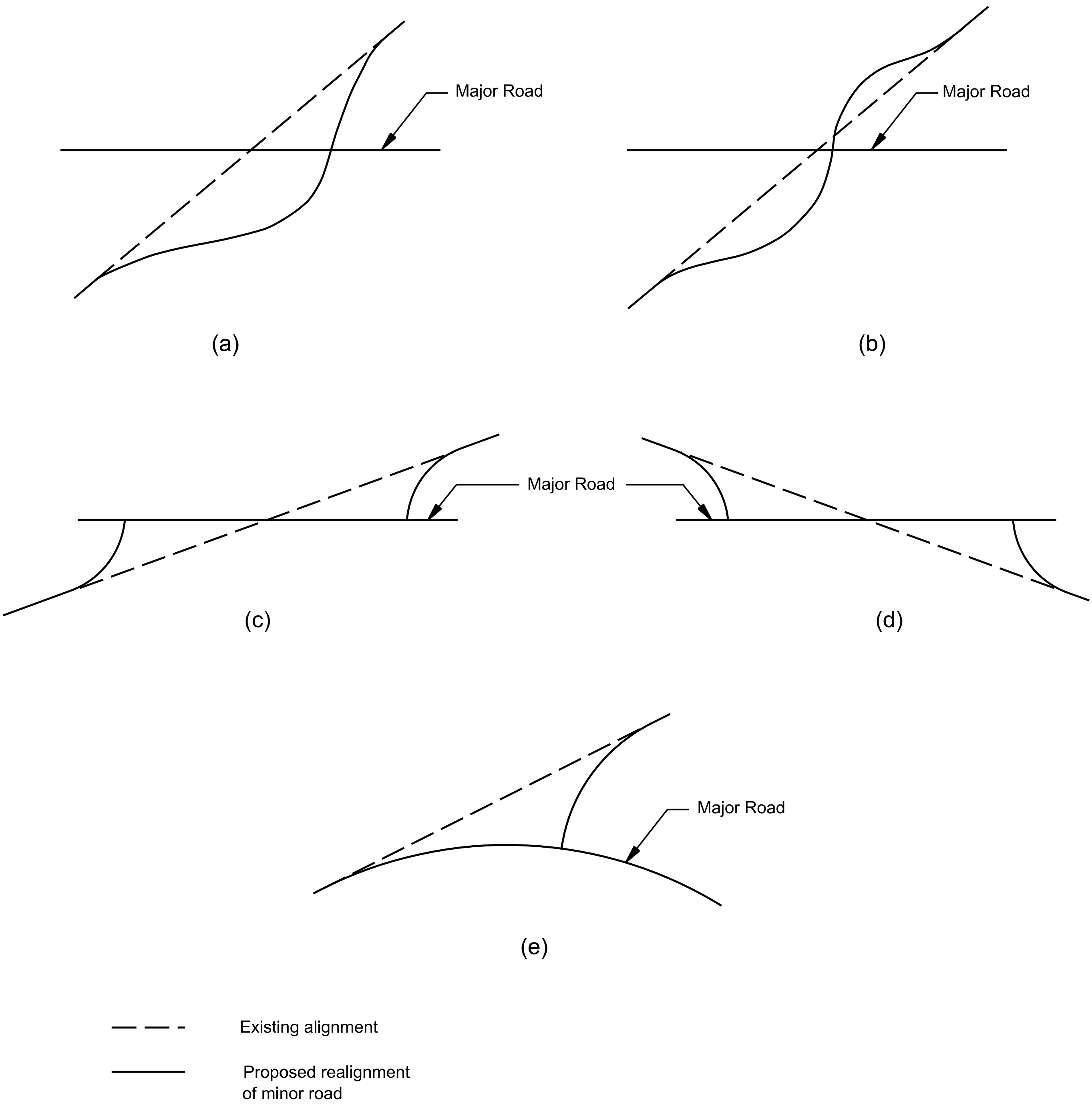


Figure 10-11 illustrates various angles of intersections and potential improvements to the alignment. Avoid using short-radius curves or unnatural travel paths near the intersection simply to reduce the intersection skew.

Figure 10-11: Realignment of Intersections



Note: *Where there are high volumes of left turns from the major road, avoid using the offset intersection alignment illustrated in (c).*

Source: (1)

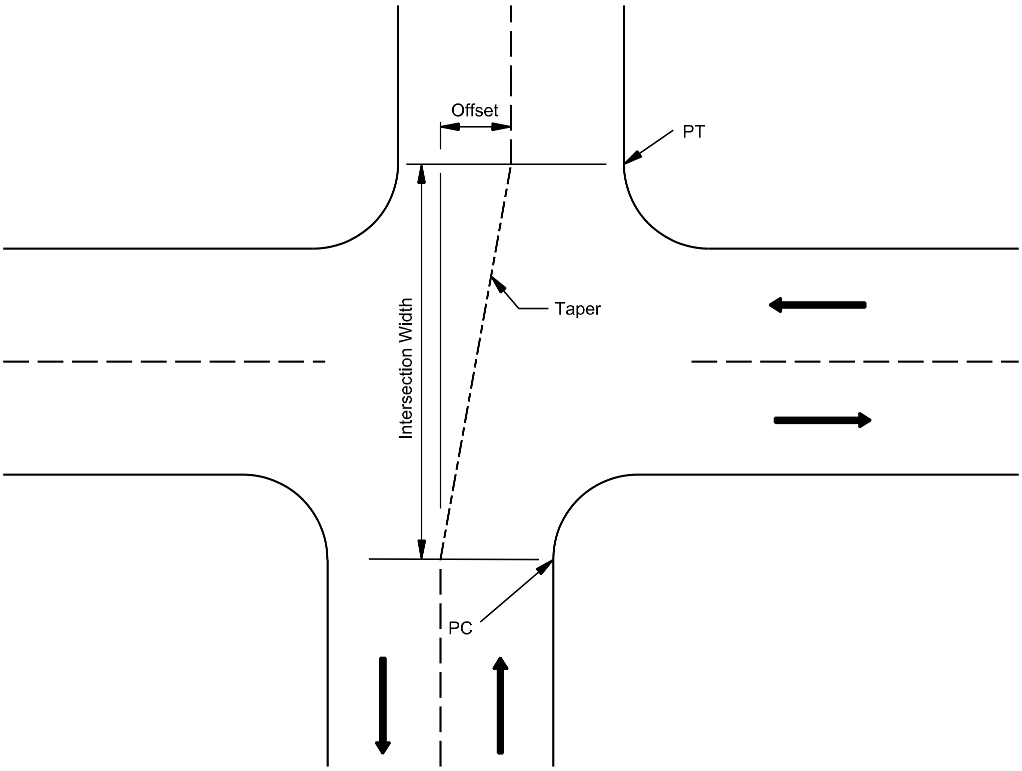
10.2.6.2 Offset Intersections

In general, 4-leg intersections should be designed so that opposing approaches line up with each other (i.e. there is no offset between opposing approaches). However, this is not always practical. Figure 10-12 presents a diagram of an intersection with an offset between opposing approaches. Because of possible conflicts with overlapping turning vehicles, offset intersections should only be allowed to remain on low-volume approaches. The following criteria will apply for offset intersection approaches:

1. Maximum Offset. The maximum offset is determined from the application of a taper equal to 0.6V:1 applied to the intersection width, where V is the design speed in kilometres per hour; see Figure 10-12. In restricted locations and where V ≤ 70 km/h, the applied taper may be $V^2/155$. V is selected as follows:

- V = 30 km/h for stop-controlled approaches.
- V = the roadway design speed for the free-flowing approaches at a stop-controlled intersection.
- V = the roadway design speed for the offset approaches at a signalised intersection.

Figure 10-12: Offset Intersection



Notes:

1. *Desirable taper rate is 0.6V:1, where V = design speed in km/h.*
2. *See discussion in Section 10.2.6.2 for more information.*

2. Turning Conflicts. Evaluate the entire intersection for conflicts that may result from turning vehicles at an offset intersection. For example, offsets where the "jog" is to the left may result in significant interference between simultaneous left-turning vehicles.

3. Evaluation Factors. In addition to potential vehicular conflicts, the designer should evaluate the following at existing or proposed offset intersections:

    - traffic volumes;
    - type of traffic control;
    - impact on all turning manoeuvres;
    - intersection geometrics (e.g. sight distance, curb/pavement edge radii); and
    - crash history at existing intersections.

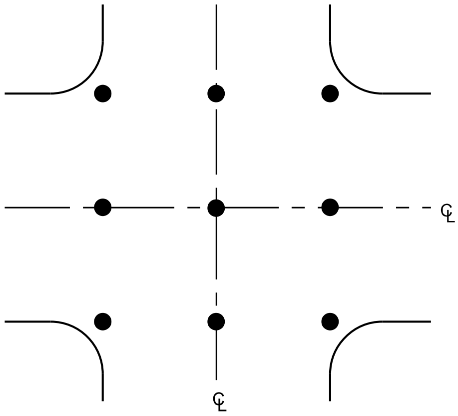
Where existing offset intersections are being considered to remain, the designer should coordinate the intersection design and traffic control requirements with the client.

10.2.7 Profiles

The design should avoid combinations of grade lines that make vehicular control difficult at intersections. To accomplish this, check the profile and edges for all roadway approaches to and through the intersection. Figure 10-13 identifies the minimum locations that should be checked. The gradients of intersecting roadways should be as flat as practical on those approaches that will be used for storage of stopped vehicles. This is commonly referred to as the storage space or storage platform. The designer should consider the following:

1. Gradients. Intersection gradients should be less than 3% on roadways. Approach gradients of 3% or steeper will require correction of certain design elements to produce operating conditions equivalent to those on level roadways (e.g. stopping sight distances, deceleration lengths). However, any gradient through the intersection must reflect the practicalities of matching the basic profiles of the intersecting roadways and shoulders. On important side roads, the storage platform gradient should be a minimum of 1% and a maximum of 2% draining away from the mainline roadway. Maintain this gradient through the expected storage distance on that leg. At a minimum, provide the storage platform gradient on the side road for a distance of 15 m to 30 m beyond the edge of the mainline travelled way or to the ditch line of an arterial or collector road.

Figure 10-13: Intersection Gradient Check Points



2. Local Roads. For local roads and entrances to the mainline roadway, provide a profile that will drain away from the mainline roadway. Where a local facility (e.g. local road, low-volume urban street) intersects a roadway on a tangent section, the side-road storage platform gradient may be a maximum of 4% draining away from the roadway.

3. Grade Lines. The principals for coordinating the horizontal and vertical alignment discussed in Chapter 6 “Vertical Alignment” are also applicable to vertical profiles through intersections. In addition, do not place intersections on or near crest vertical curves unless the vertical curve is flat enough for the intersection pavement to be seen assuming decision sight distance.

10.2.8 Design Vehicles

10.2.8.1 Types

The design vehicle affects the radius returns, left-turn radii, lane widths, median openings, turning roadways, and sight distances at an intersection. The basic design vehicles used in the Emirates for intersection design are:

- P — Passenger car; includes vans and pickup trucks.
- SU-9 — Single unit truck; includes the typical fire truck.
- SU-12 — Single-unit truck (3 axles).
- *WB-15 — Tractor/semitrailer combination with an overall wheelbase of 15.2 m.
- *WB-20 — Tractor/semitrailer combination with an overall wheelbase of 19.4 m.
- *WB-33D — Tractor/double-trailer combination with an overall wheelbase of 33.3 m.

* These design vehicles only apply to rural areas. For urban areas, these vehicles may need to be accommodated as a control vehicle.

Chapter 2 “General Design Criteria and Control” provides the vehicular dimensions for each of the design vehicles.

10.2.8.2 Selection

Urban intersection/junction design requires consideration of at least four design vehicles:

1. Speed Control Vehicle. For all street types, this is a passenger car, and it is used to determine maximum vehicle speeds at turns.

2. Design Vehicle. A vehicle that must be regularly accommodated without encroachment into the opposing traffic lanes. This vehicle may vary depending on the street type and context.

3. Control Vehicle. A vehicle that infrequently uses a facility and must be accommodated. For these vehicles, encroachment into the opposing traffic lanes, multiple-point turns, or minor encroachment into the roadside is acceptable.

4. Non-Motorised Vehicles. On priority cycle routes, special design consideration is necessary for cycles; see Section13.2.4.

Table 10-5 presents the recommended design vehicles at intersections based on the functional classification of the intersecting roadways from which the vehicle is turning. The design vehicle should be a WB-20. In addition to Table 10-5, consider the following guidelines when selecting a design vehicle:

1. Minimum Designs. The SU-9 and/or SU-12 design vehicles are generally the smallest vehicles used in the physical design of roadway intersections. This design reflects that, even in residential areas, garbage trucks, fire trucks, delivery trucks, and school buses will be negotiating turns with some frequency. Urban intersections are typically designed using the largest design vehicles that are expected to use that intersection with significant frequency.

2. Recreational Areas. Recreational areas typically will be designed using the SU-12 design vehicle. This reflects that service vehicles are typically required to maintain the recreational area. Under some circumstances, the SU-9 may be the appropriate design vehicle.

3. Design versus Control Vehicle. The selected design vehicle should be able to make turns without encroachment into adjacent or opposing lanes. The control vehicle should physically be able to make the turn with some encroachment into adjacent and/or opposing lanes.

4. Turning Template. Check the intersection design and layout with an approved computer simulated turning template program.

10.2.9 Pedestrians and Cyclists

Safe and convenient movement of pedestrians and cyclists through the intersection must be considered in the design of an urban intersection. However, this often causes conflicting objectives in the overall design of an intersection. Wider intersection designs to accommodate the design vehicle significantly increase the crossing distance for pedestrians. At signalised intersections, longer crossing times and conflicts with turning vehicles can significantly affect the overall capacity of the intersection. To reduce these problems, the geometric layout of the intersection may need to be revised, refuge islands included within the intersection, special turn lanes added for cyclists, or other factors included in the design.

Chapter 13 “Pedestrian and Cyclist Facilities” and the Abu Dhabi *Walking and Cycling Master Plan* (23) provide guidance for accommodating cycle lanes and pedestrians through an intersection.

10.2.10 Signing and Pavement Markings

Use the current edition of the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) to design the signing and pavement markings at intersections.

10.2.11 Drainage

Evaluate the profile and transitions at all intersections for impacts on drainage. This is especially important for channelized intersections on curves and grades. This may require the designer to check superelevation transition lengths to ensure flat sections are minimised. Low points on approach roadway profiles should be beyond a raised corner island to prevent water from being trapped and causing ponding. See the Abu Dhabi *Road Drainage Manual* (26) for guidance.

Table 10-5: Selection of Design Vehicle at Intersections (Functional Classification)

For Turn Made		Design Vehicle
From	**Onto**	
Freeway Ramp	Other facilities	WB-20*
Other Facilities	Freeway ramp	WB-20*
Rural Arterial	Rural Arterial Truck route Rural Collector Rural Local Local (residential)	WB-20* WB-20* WB-15 WB-15 SU-12**
Truck Route	Rural Arterial Truck Route Rural Collector Rural Local Local (residential)	WB-20* WB-20* WB-15 WB-15 SU-12**
Rural Collector	Rural Arterial Truck route Rural Collector Rural Local Local (residential)	WB-15 WB-15 WB-15 WB-15 SU-12**
Rural Local	Rural Arterial Truck route Rural Collector Rural Local Local (residential)	WB-15 WB-15 WB-15 SU-12** SU-9
Boulevard or Avenue	All	Design Vehicle: WB-12 Control Vehicle: WB-33D
Street	All	Design Vehicle: SU-9 but City-Bus M on public bus routes Control Vehicle: Smeal Aerial RM 100 Fire Truck***
Access Lane	All	Design Vehicle: SU-9 Control Vehicle: SU-9

** With encroachment, a WB-33D vehicle should physically be able to make the turn.*
***With encroachment, a WB-15 vehicle should physically be able to make the turn.*
****See the DMT Urban Street Design Manual* (4) *for vehicle dimensions.*

10.3 Corner Radii

Corner radii treatments for intersections are important design elements in that they influence the operation (e.g. turning speeds), safety (e.g. pedestrians), and construction costs of the intersection. The designer must ensure that the proposed design is compatible with the expected intersection operations.

10.3.1 Design for Right-Turning Vehicles

The following sections present the basic parameters the designer needs to consider in determining the proper pavement edge/curb line for right-turning vehicles.

10.3.1.1 Design Vehicle

Section 10.2.8 discusses the selection of the applicable design vehicle for different intersections. These vehicles are used to determine the pavement edge or curb line. Note that the design vehicle will determine the turning width, vehicular path width or swept-path width. The assumed speed of the vehicle is less than 15 km/h. Note that an intersection design to accommodate a large semi-trailer combination will allow faster turns for passenger cars. The designer is responsible for ensuring if these higher turning speeds are safe for pedestrians and cyclists. If not, smaller control radii or separate signal phases may be required.

10.3.1.2 Inside Clearance

Desirably, the selected design vehicle will make the right turn while maintaining approximately a 600 mm clearance from the pavement edge or face of curb.

10.3.1.3 Encroachment

To determine the amount of acceptable encroachment, the designer should evaluate several factors. These would include traffic volumes, one-way or two-way operations, urban/rural location, and the type of traffic control. For turns made onto local facilities, desirably the selected design vehicle will not encroach into the opposing travel lanes. However, this is neither always practical nor cost effective in urban areas. The designer must evaluate these encroachment conditions against pedestrian and cyclists needs, construction, and right-of-way impacts. If these impacts are significant and if through and/or turning volumes are relatively low, the designer may consider accepting some encroachment of the design vehicle into opposing lanes; see Figure 10-14.

The encroachment allowed into adjacent lanes of the road or street onto which the turn is made will depend on the following:

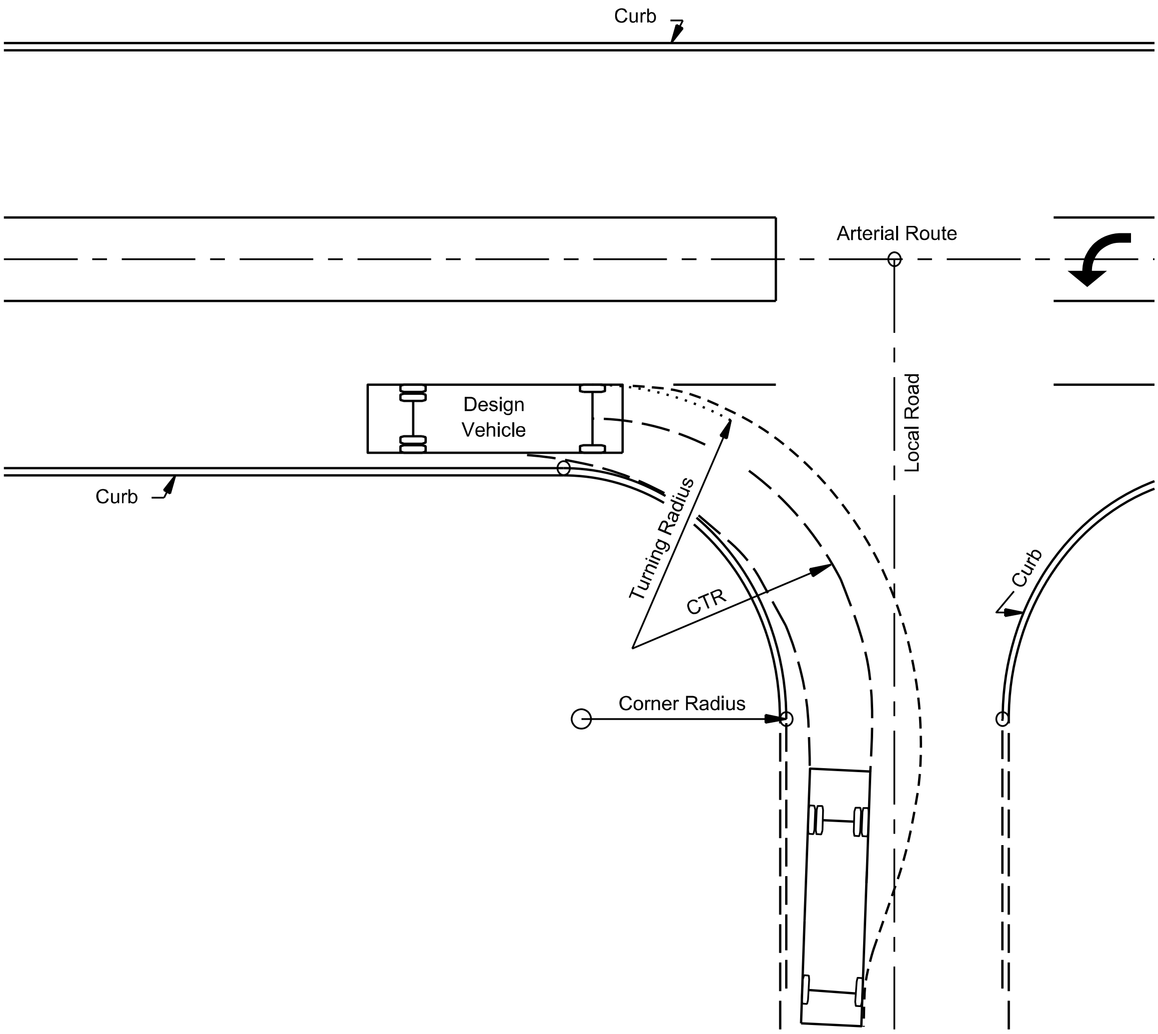
1. <u>Urban</u>. No encroachment for the design vehicle should be allowed into opposing lanes for a right-turning vehicle from a side road or street onto Boulevard or Avenue. Some encroachment may be allowed for emergency vehicles.

2. <u>Rural</u>. For rural intersections, the selected design vehicle should not encroach into the opposing lanes of traffic.

3. <u>Multilane Roadways</u>. If there are two or more lanes of traffic in the same direction on the road onto which the turn is made, the selected design vehicle can occupy both travel lanes. Desirably, the right-turning vehicle will be able to make the turn while remaining entirely in the right through lane; see Figure 10-15.

4. Truck Routes. Check all intersections of two designated truck routes to ensure a WB-20 (or WB-33D) design vehicle can physically make the right turn without backing up and without impacting curbs, parked cars, traffic control devices, or any other obstructions, regardless of the selected design vehicle or allowable encroachment.

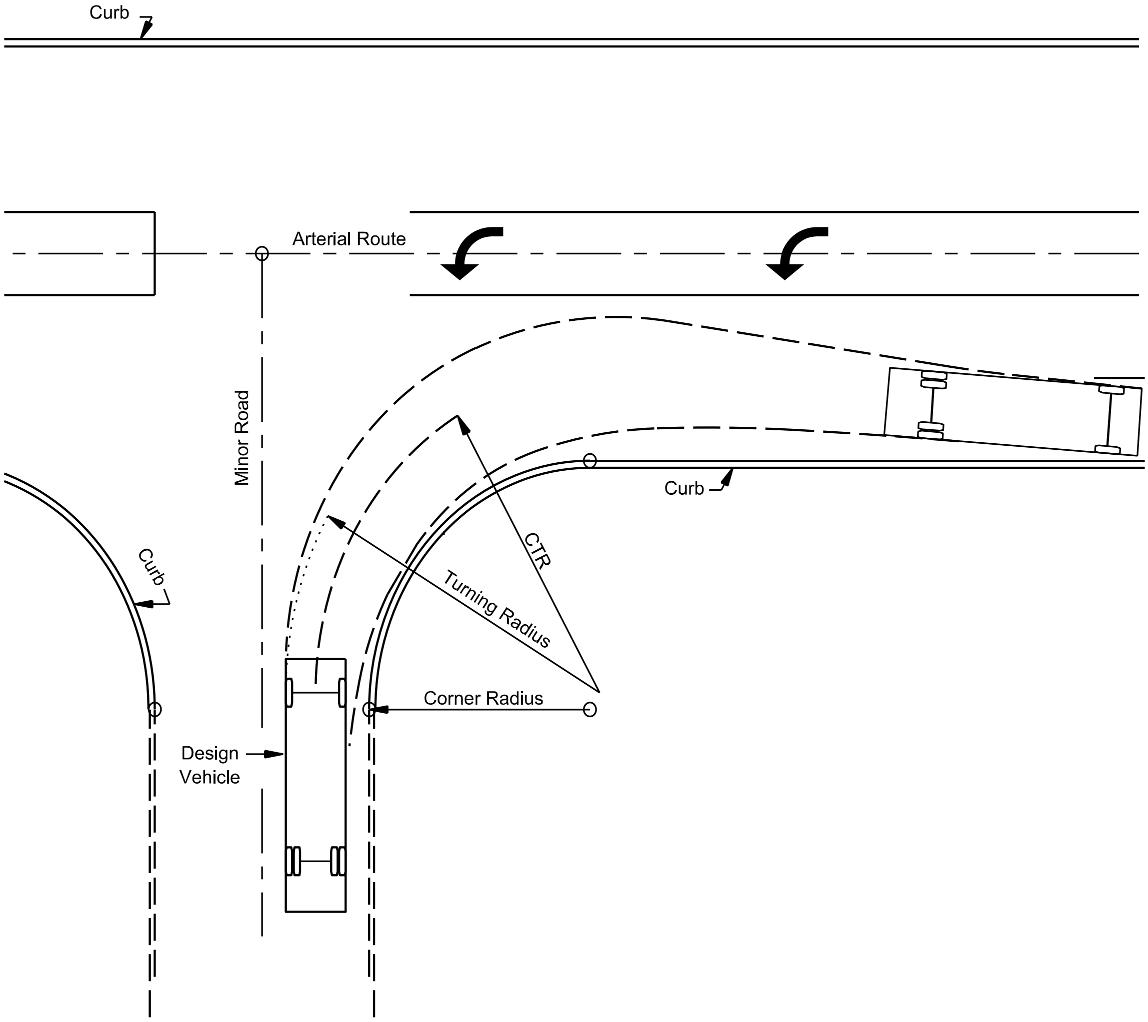
Figure 10-14: Allowable Encroachment on a Local Road



Notes:

1. *Figure illustrates restricted cross section on arterial route and local road.*
2. *Turning radius of truck at centreline of front axle = CTR.*

Figure 10-15: Allowable Turning Encroachment onto a Multilane Road



Notes:

1. *Figure indicates restricted cross section on arterial route.*
2. *Turning radius of truck at centreline of front axle = CTR.*

10.3.1.4 Parking Lanes/Shoulders

At many intersections, parking lanes and/or shoulders will be available on one or both approach legs. This additional roadway width may be carried through the intersection. The following will apply:

1. Parking Lanes. Under restricted conditions, the designer may take advantage of shoulder and/or parking lanes to ease the problems of large vehicles turning right at intersections with small radius returns. It will be necessary to restrict the parking a significant distance from the intersection. This area should be delineated with pavement markings. Parking should be restricted from the intersection according to the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

2. Paved Shoulders. At rural intersections, it may be preferable to continue a paved shoulder throughout the radius return. If a shoulder width transition is required, design it according to Figure 10-16(a).

3. Curbing. If certain conditions (e.g. drainage requirements, restricted right-of-way, greater delineation, or the desire to minimise off-tracking) at rural intersections warrant the use of curbing along the radius return at rural intersections, terminate the curbing at the shoulder edge and transition the curb height as indicated in Figure 10-16(b). Where posted speeds are 80 km/h or greater, use a sloping curb.

   In urban areas, provide a 100 mm high sloping curb for the corner radii and an adequate obstruction free area to allow emergency vehicles to mount the corner curbing.

10.3.1.5 Pedestrian Considerations

The larger the right-turning radius, the farther pedestrians must walk across the street. This is especially important to persons with disabilities. Therefore, the designer must consider the number and type of pedestrians using an intersection when determining the edge of pavement or curb line design. This may lead to a decision to design a right-turn corner island (small or intermediate) for use as a pedestrian refuge. Where speeds and volumes warrant, the designer may want to consider provided protected pedestrian refuge areas.

10.3.1.6 Types of Right-Turn Designs

Once the designer has determined the basic right-turning parameters (e.g. design vehicle, amount of allowable encroachment, inside clearance), it will be necessary to select the type of turning design for the curb return or pavement edge that will meet these criteria and will fit the intersection constraints. Most urban intersections are designed using a simple radius corner island. Where large trucks are expected to be turning with relative frequency, consider providing a simple radius with tapers or a three-centred compound radius (see Figure 10-17). The simple radius with tapers and the three-centre radii better fit the off-tracking path of large trucks and will reduce the overall intersection width and pavement area.

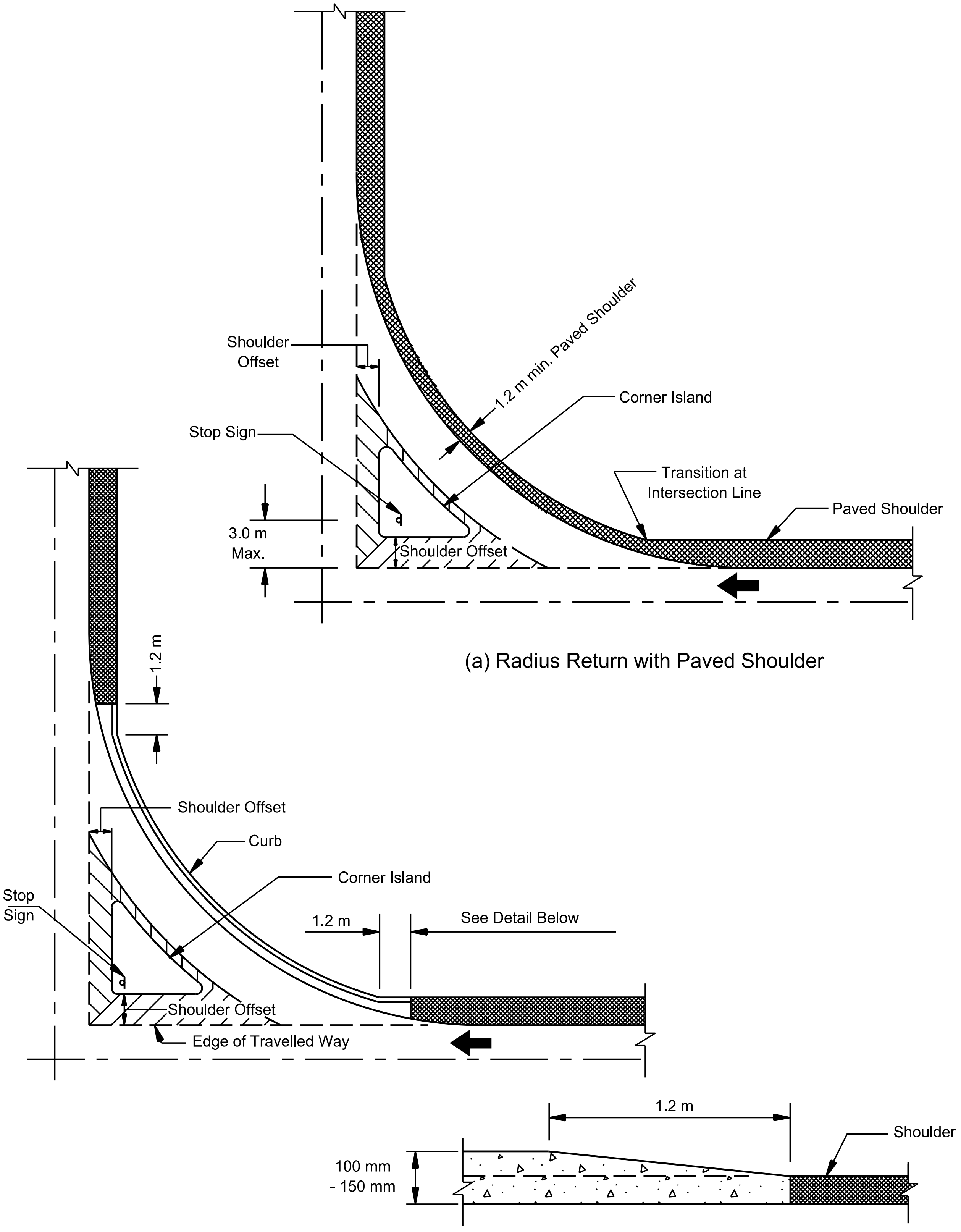
10.3.1.7 Stop Bar Locations

Stop bar locations should be checked against the criteria in the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) at wide throat intersections (e.g. the typical placement for the stop bar should be 1 m to 3 m from the crossing roads edge of travelled way). This is especially important where no corner island is used.

10.3.1.8 Turning Template(s)

The designer should use the applicable turning templates or computer simulated turning program to design and check the corner radii design. Provide the DMT with a copy of output from the turning template program.

Figure 10-16: Shoulder/Curb Radius Return Transitions for Rural Intersections

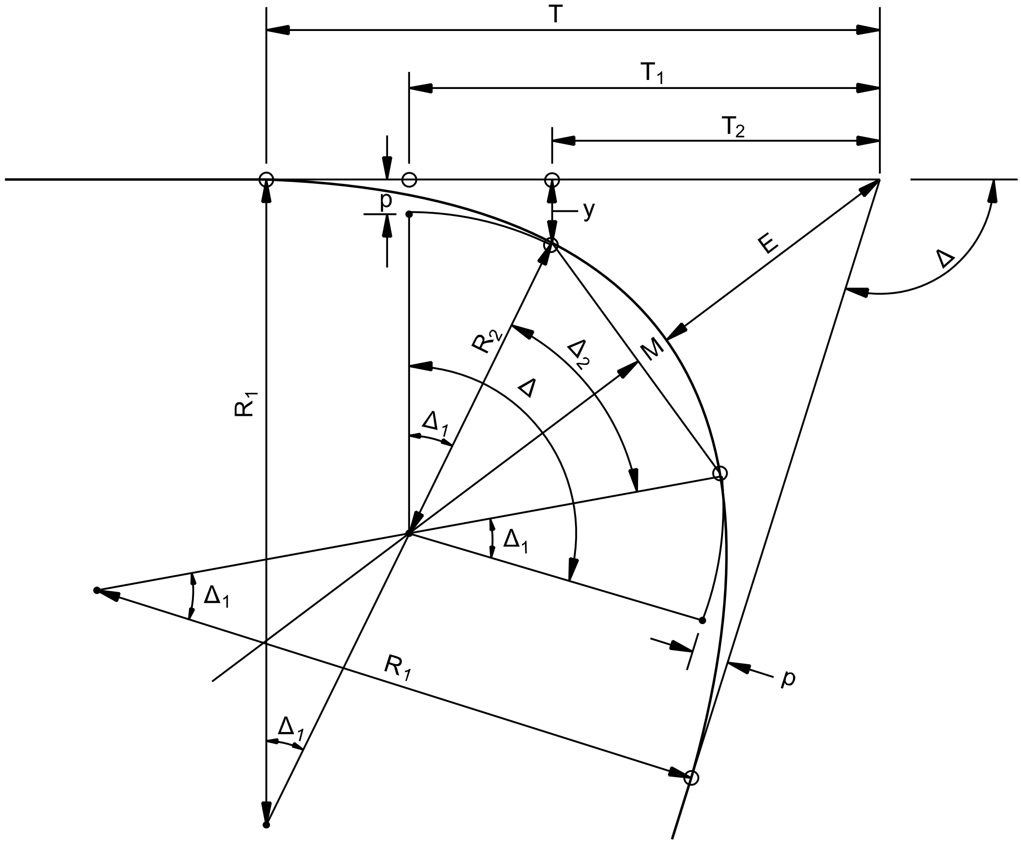


(a) Radius Return with Paved Shoulder

Detail of Curb Height Transition

(b) Radius Return with Curb

Figure 10-17: Three-Centred Compound Radii



Given: R_1, R_2, Δ_1, and p

$$T_1 = (R_2 + p)\tan\frac{\Delta}{2}$$

$$E = \frac{R_2 + p}{\cos(\Delta/2)} - R_2$$

$$\Delta_1 = \cos^{-1}\left[\frac{R_1 - R_2 - p}{R_1 - R_2}\right]$$

$$M = R_2 - (R_2 \cos(\Delta/2 - \Delta_1))$$

$$T = T_1 + (R_1 - R_2)\sin\Delta_1$$

$$y = (R_2 + p) - R_2 \cos\Delta_1$$

$$T_2 = T_1 - R_2 \sin\Delta_1$$

$$\Delta_2 = \Delta - 2\Delta_1$$

$$\Delta = 2\Delta_1 + \Delta_2$$

Note: "p" is the offset location between the interior curve (extended) to a point where it becomes parallel with the tangent line.

10.3.1.9 Minimum Edge-of-Travelled Way Designs

Where it is appropriate to provide for turning vehicles within minimum space, as at unchannelized intersections, the corner radii should be based on the minimum turning path of the selected design vehicles. The swept path widths, which are slightly greater than the minimum paths of nearly all vehicles in the class represented by each design vehicle, are the minimum paths attainable at speeds equal to or less than 15 km/h and consequently offer some flexibility for driver behaviour. These turning paths of the design vehicles are considered satisfactory as minimum designs. Table 10-6 and Table 10-7 summarise minimum corner radii designs based on the design vehicle and angle of turn.

10.3.1.10 Urban Corner Radii Guidelines

Corner design in urban areas deserves special consideration as it directly impacts pedestrian crossing distances and vehicle turning speeds. Smaller corner radii facilitate direct pedestrian desire lines and discourage speeding, creating intersections that are safer for pedestrians and cyclists. In addition, consider the following:

1. Residential Streets. In low-density residential areas, a maximum corner radius of 5 m should be used at intersections. However, 7.5-m radius may be considered in particular circumstances to accommodate frequent usage by large vehicles (e.g. buses and delivery vehicles). Ensure the width of the travelled way is not less than 6 m in total (excluding on-street parking) and no refuge islands are provided on Streets.

2. City, Town and Commercial Contexts. Typically, use a corner radius of 7.5 m.

3. Industrial Contexts. Corner radii can be increased to 12 m to accommodate larger numbers of trucks and servicing vehicles.

4. Turning Speeds. Design turns so that vehicles do not turn faster than 15 km/h.

5. Effective Turning Radius. Calculate the effective turning radius and available space for turning, including the space for on-street parking, cycle lanes, and all travel lanes on the receiving street (not just the nearest lane).

6. Computer Turning Templates. Use computer software for designing and testing turning radii. Check the built-in tolerances so that they do not cause overly large radii.

7. Civil Defence Department Criteria. Corner radii can be increased to meet the operational requirements of the Civil Defence Department at specific locations. In all cases, use the minimum radius that meets Civil Defence Department requirements.

8. Encroachment. Check the turning movement for large vehicles to ensure that these vehicles remain within the same lane during turning movements, where practical. Occasional larger vehicles, including emergency vehicles, may encroach into opposing lanes and/or drive over corners to complete the turn on low volume Streets and Access Lanes.

Table 10-6: Edge-of-Travelled-Way Designs for Turns at Intersections — Simple Curve Radius with Taper

Angle of Turn (degrees)	Design Vehicle	Simple Curve Radius (m)	Simple Curve Radius With Taper		
			Radius (m)	Offset (p) (m)	Taper L:T
30	P	18	—	—	—
	SU-9	30	—	—	—
	SU-12		—	—	—
	WB-12	45	—	—	—
	WB-15	60	—	—	—
	WB-20	116	67	1.0	15:1
	WB-33D	145	77	1.1	20:1
45	P	15	—	—	—
	SU-9	23	—	—	—
	SU-12	35	—	—	—
	WB-12	36	—	—	—
	WB-15	53	36	0.6	15:1
	WB-20	76	43	1.3	15:1
	WB-33D	—	60	1.3	20:1
60	P	12	—	—	—
	SU-9	18	—	—	—
	SU-12	30	—	—	—
	WB-12	28	—	—	—
	WB-15	45	29	1.0	15:1
	WB-20	60	43	1.3	15:1
	WB-33D	—	54	1.3	20:1
75	P	11	8	0.6	10:1
	SU-9	17	14	0.6	10:1
	SU-12	27	18	0.6	10:1
	WB-12	—	18	0.6	15:1
	WB-15	—	20	1.0	15:1
	WB-20	—	43	1.3	20:1
	WB-33D	—	42	1.7	20:1
90	P	9	6	0.8	10:1
	SU-9	15	12	0.6	10:1
	SU-12	24	14	1.2	10:1
	WB-12	—	14	1.2	10:1
	WB-15	—	18	1.2	15:1
	WB-20	—	37	1.3	30:1
	WB-33D	—	35	0.9	15:1
105	P	—	6	0.8	8:1
	SU-9	—	11	1.0	10:1
	SU-12	—	14	1.2	10:1
	WB-12	—	12	1.2	10:1
	WB-15	—	17	1.2	15:1
	WB-20	—	35	1.0	15:1
	WB-33D	—	28	2.8	20:1

Source: (1)

Table 10.6: Edge-of-Travelled-Way Designs for Turns at Intersections — Simple Curve Radius with Taper
(*Continued*)

Angle of Turn (degrees)	Design Vehicle	Simple Curve Radius (m)	Simple Curve Radius With Taper		
			Radius (m)	Offset (p) (m)	Taper L:T
120	P	—	6	0.6	10:1
	SU-9	—	9	1.0	10:1
	SU-12	—	11	1.8	8:1
	WB-12	—	11	1.5	8:1
	WB-15	—	14	1.2	15:1
	WB-20	—	31	1.6	15:1
	WB-33D	—	26	2.8	20:1
135	P	—	6	0.5	10:1
	SU-9	—	9	1.2	10:1
	SU-12	—	12	1.2	8:1
	WB-12	—	9	2.5	15:1
	WB-15	—	12	2.0	15:1
	WB-20	—	25	1.6	20:1
	WB-33D	—	25	2.6	20:1
150	P	—	6	0.6	10:1
	SU-9	—	9	1.2	8:1
	SU-12	—	11	2.1	8:1
	WB-12	—	9	2.0	8:1
	WB-15	—	11	2.1	6:1
	WB-20	—	19	3.1	10:1
	WB-33D	—	20	4.6	10:1
180	P	—	5	0.2	20:1
	SU-9	—	9	0.5	10:1
	SU-12	—	11	2.0	10:1
	WB-12	—	6	3.0	5:1
	WB-15	—	8	3.0	5:1
	WB-20	—	16	4.2	10:1
	WB-33D	—	17	6:1	10:1

Table 10-7: Edge-of-Travelled-Way Designs for Turns at Intersections — Three-Centred Curves

Angle of Turn (degrees)	Design Vehicle	Three-Centred Compound		Three-Centred Compound	
		Curve Radii (m)	Symmetric Offset (m)	Curve Radii (m)	Asymmetric offset (m)
30	P	—	—	—	—
	SU-9	—	—	—	—
	SU-12	—	—	—	—
	WB-12	—	—	—	—
	WB-15	—	—	—	—
	WB-20	140-53-140	1.2	91-53-168	0.6 – 1.4
	WB-33D	168-76-168	1.5	76-61-198	0.5 – 2.1
45	P	—	—	—	—
	SU-9	—	—	—	—
	SU-12	—	—	—	—
	WB-12	—	—	—	—
	WB-15	60-30-60	1.0	36-43-150	1.0 – 2.6
	WB-20	140-53-140	1.2	76-38-183	0.3 – 1.8
	WB-33D	168-61-168	1.5	61-52-198	0.5 – 2.1
60	P	—	—	—	—
	SU-9	—	—	—	—
	SU-12	—	—	—	—
	WB-12	—	—	—	—
	WB-15	60-23-60	1.7	60-23-84	0.6 – 20
	WB-20	122-30-122	2.4	76-38-183	0.3 – 1.8
	WB-33D	198-46-198	1.7	61-43-183	0.5 – 2.4
75	P	30-8-30	0.6	—	—
	SU-9	36-14-36	0.6	—	—
	SU-12	61-11-61	1.5	18-14-61	0.3 – 1.4
	WB-12	36-14-36	1.5	36-14-60	0.6 – 2.0
	WB-15	45-15-45	2.0	45-15-69	0.6 – 3.0
	WB-20	128-23-128	3.0	61-24-183	0.3 – 3.0
	WB-33D	213-38-213	2.0	46-34-168	0.5 – 3.5
90	P	30-6-30	0.8	—	—
	SU-9	36-12-36	0.6	—	—
	SU-12	61-9-61	2.1	18-14-61	0.3 – 1.4
	WB-12	36-12-36	1.5	36-12-60	0.6 – 2.0
	WB-15	55-18-55	2.0	36-12-60	0.6 – 3.0
	WB-20	134-20-134	3.0	61-21-183	0.3 – 3.4
	WB-33D	213-34-213	2.0	30-29-168	0.6 – 3.5
105	P	30-6-30	0.8	—	—
	SU-9	30-11-30	1.0	—	—
	SU-12	61-11-61	1.8	18-12-58	0.5 – 1.8
	WB-12	30-11-30	1.5	30-17-60	0.6 – 2.5
	WB-15	55-14-55	2.5	45-12-64	0.6 – 3.0
	WB-20	152-15-152	4.0	61-20-183	0.3 – 3.4
	WB-33D	213-29-213	2.4	46-24-152	0.9 – 4.6

Source: (1)

Table 10.7: Edge-of-Travelled-Way Designs for Turns at Intersections — Three-Centred Curves
(*Continued*)

Angle of Turn (degrees)	Design Vehicle	Three-Centred Compound		Three-Centred Compound	
		Curve Radii (m)	**Symmetric Offset (m)**	**Curve Radii (m)**	**Asymmetric Offset (m)**
120	P	30-6-30	0.6	—	—
	SU-9	30-9-30	1.0	—	—
	SU-12	61-11-61	1.8	18-12-58	0.5 – 1.5
	WB-12	36-9-36	2.0	30-9-55	0.6 – 2.7
	WB-15	55-12-55	2.6	45-11-67	0.6 – 3.6
	WB-20	168-14-168	4.6	61-18-183	0.6 – 3.8
	WB-33D	213-26-213	2.7	46-21-152	2.0 – 5.3
135	P	30-6-30	0.5	—	—
	SU-9	30-9-30	1.2	—	—
	SU-12	61-12-61	1.2	18-12-55	0.5 – 1.5
	WB-12	36-9-36	2.0	30-8-55	1.0 – 4.0
	WB-15	48-11-48	2.7	40-9-56	1.0 – 4.3
	WB-20	168-14-168	5.0	61-18-183	0.6 – 3.8
	WB-33D	213-21-213	3.8	46-20-152	2.1 – 5.6
150	P	23-6-23	0.6	—	—
	SU-9	30-9-30	1.2	—	—
	SU-12	61-11-61	2.0	18-12-61	0.3 – 1.4
	WB-12	30-9-30	2.0	28-8-48	0.3 – 3.6
	WB-15	48-11-48	2.1	36-9-55	1.0 – 4.3
	WB-20	168-14-168	5.8	61-17-183	2.0 – 5.0
	WB-33D	213-20-213	4.6	61-20-152	2.7 – 5.6
180	P	15-5-15	0.2	—	—
	SU-9	30-9-30	0.5	—	—
	SU-12	46-11-46	1.9	15-11-40	1.7 – 2.1
	WB-12	30-6-30	3.0	26-6-45	2.0 – 4.0
	WB-15	40-18-40	3.0	30-8-55	2.0 – 4.0
	WB-20	183-14-183	6.2	30-17-122	1.8 – 4.6
	WB-33D	213-17-213	6.1	61-18-152	3.0 – 6.4

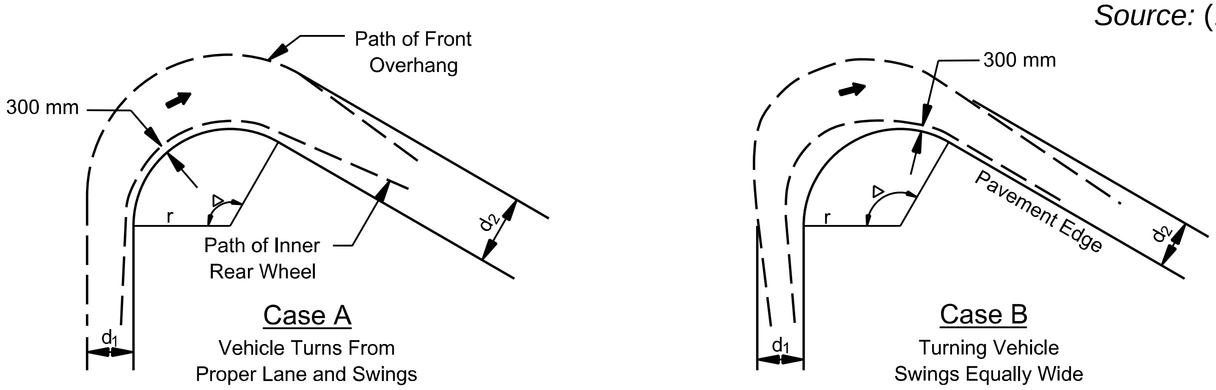
10.3.1.11 Effect of Curb Radii on Turning Paths

Table 10-8 shows the effect of the angle of intersection on turning paths of various design vehicles on streets without parking lanes. The dimensions d_1 and d_2 are the widths occupied by the turning vehicle on the major street and cross street, respectively, while negotiating turns through various angles. Both dimensions are measured from the right-hand curb to the point of maximum overhang. These widths, shown for various angles of turn and curb radii and for two types of manoeuvres, generally increase with the angle of turn.

Table 10-8: Cross Street Width Occupied by Turning Vehicle for Various Angles of Intersection and Curb Radii

Angle of Intersection (∆)	Design Vehicle	d_2 for Cases A and B where:									
		R=4.5 m		R=6.0 m		R=7.5 m		R=9.0 m		R=12.0 m	
		A m	B m	A m	B m	A m	B m	A m	B m	A m	B m
30°	SU-9	4.3	4.0	4.3	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	SU-12	4.9	4.6	4.6	4.3	4.6	4.3	4.6	4.3	4.3	4.0
	BUS	6.7	5.2	5.8	5.2	5.8	5.2	5.8	5.2	5.5	5.2
	WB-12	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	WB-19	—	—	—	—	—	—	—	—	8.2	5.2
	WB-20	—	—	—	—	—	—	—	—	8.5	5.5
60°	SU-9	5.8	4.9	5.8	4.9	5.2	4.6	4.9	4.6	4.3	4.3
	SU-12	7.3	5.8	5.8	5.5	5.8	5.2	5.5	4.9	5.2	4.6
	BUS	8.5	6.4	7.9	6.1	7.3	6.1	7.0	5.8	6.7	5.5
	WB-12	7.3	5.8	6.7	5.8	6.4	5.8	5.8	5.5	5.2	4.9
	WB-19	—	—	—	—	—	—	—	—	9.1	6.7
	WB-20	—	—	—	—	—	—	—	—	11.3	7.3
90°	SU-9	7.9	6.1	7.0	5.5	5.8	4.9	5.2	4.6	4.0	4.0
	SU-12	8.8	6.4	7.9	5.8	6.7	5.8	5.8	5.2	4.6	4.6
	BUS	11.6	7.0	10.0	6.7	9.1	6.7	7.6	6.4	6.7	5.5
	WB-12	9.4	6.7	8.2	6.4	7.0	6.4	5.8	5.5	5.2	4.9
	WB-19	—	—	—	—	—	—	—	—	11.9	7.0
	WB-20	—	—	—	—	—	—	—	—	11.9	7.6
120°	SU-9	10.4	6.7	8.2	5.8	6.4	5.5	5.2	4.9	4.0	4.0
	SU-12	12.2	7.6	10.4	7.0	8.2	6.1	6.7	5.5	4.9	4.6
	BUS	14.0	8.5	12.2	7.6	9.8	7.0	7.9	5.8	5.8	5.5
	WB-12	11.3	7.0	8.8	6.7	7.3	6.7	5.8	5.5	5.2	4.9
	WB-19	—	—	—	—	—	—	—	—	7.9	6.7
	WB-20	—	—	—	—	—	—	—	—	9.1	7.0
150°	SU-9	12.2	7.6	9.8	6.4	6.7	5.8	5.2	4.9	3.6	3.6
	SU-12	15.2	9.1	12.5	7.9	10.1	7.0	7.6	5.8	2.9	4.6
	BUS	14.6	8.5	12.2	7.6	9.8	7.0	5.7	5.5	5.2	4.9
	WB-12	11.9	7.3	8.8	6.7	7.0	6.7	5.8	5.5	5.2	4.9
	WB-19	—	—	—	—	—	—	—	—	6.1	5.5
	WB-20	—	—	—	—	—	—	—	—	8.2	5.5

Source: (1)



Case A
Vehicle Turns From Proper Lane and Swings Wide on Cross Street
d_1=3.65 m d_2 is Variable

Case B
Turning Vehicle Swings Equally Wide on Both Streets
d_1 = d_2 Both Variable

d_1 = Maximum distance between the pavement edge (before the turn) and the vehicle overhang.
d_2 = Maximum distance between the pavement edge (after the turn) and the vehicle overhang.

Table 10-8 also shows that a very large radius should be used or the streets should be very wide to accommodate the longer vehicles, particularly where the central angle is greater than 90 degrees. For this reason, simple curves with taper offsets or three-centred radii are preferred to fit the paths of vehicles properly. Data are shown for simple curves. The radii for simple curves have been omitted for angles of turn greater than 90 degrees for the larger trucks for the reasons given above. However, they may be used for right-turn designs where sufficient right-of-way is available and where there is little pedestrian traffic. Table 10-8 shows the cross street width occupied by turning vehicle for various angles of intersection and curb radii.

With parking allowed along a curbed street, vehicles (except for WB-20 and larger vehicles) are able to turn without encroachment onto adjacent lanes, even where curb radii are relatively small. As shown in Figure 10-18, the SU-9 and WB-15 design vehicles are able to turn at a 4.5-m curb radius with little, if any, encroachment on adjacent approach lanes. However, parking should be restricted for a distance of at least 4.5 m in advance of the right-turning radius. The parking restriction should extend at least 4.5 m beyond the end of the radius for SU-9 design vehicles and at least 9.0 m beyond the end of the radius for the WB-15 design vehicle. The WB-15 design vehicles will encroach onto the opposing lanes in making a turn unless the turning radius is at least 7.5 m and parking is restricted at the far end of the turn for at least 12 m beyond the radius. In case such encroachment is considered undesirable, then consideration may be given to slightly increasing the road width for the turning lane.

In all cases consideration must be given for visibility to NMUs at the crossings at these junctions.

10.3.1.12 Summary

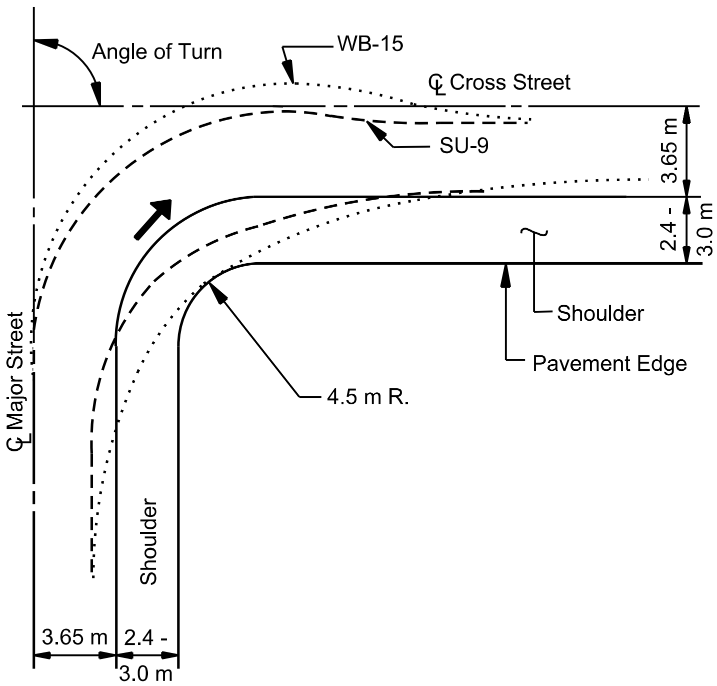
Figure 10-19 illustrates the many factors that should be evaluated in determining the proper design for right-turns movements at intersections. In summary, the following procedure applies:

1. Select the design vehicle (Section 10.3.1.1).
2. Determine the acceptable inside clearance (Section 10.3.1.2).
3. Determine the acceptable encroachment (Section 10.3.1.3).
4. Consider the benefits of any parking lanes or shoulders (Section 10.3.1.4).
5. Consider impacts on pedestrians (Section 10.3.1.5).
6. Select the type of right-turning treatment (Section 10.3.1.6).
7. Check the location of the stop bar (Section 10.3.1.7).

Check all proposed designs with the applicable vehicular turning templates or computer simulated turning template program to ensure all of the above criteria are met (Section 10.3.1.8).

Revise the design as necessary to accommodate the right-turning vehicle or determine that it is not practical to meet this design because of adverse impacts.

Figure 10-18: Effect of Curb Radii and Shoulders on Rural Right-Turning Paths



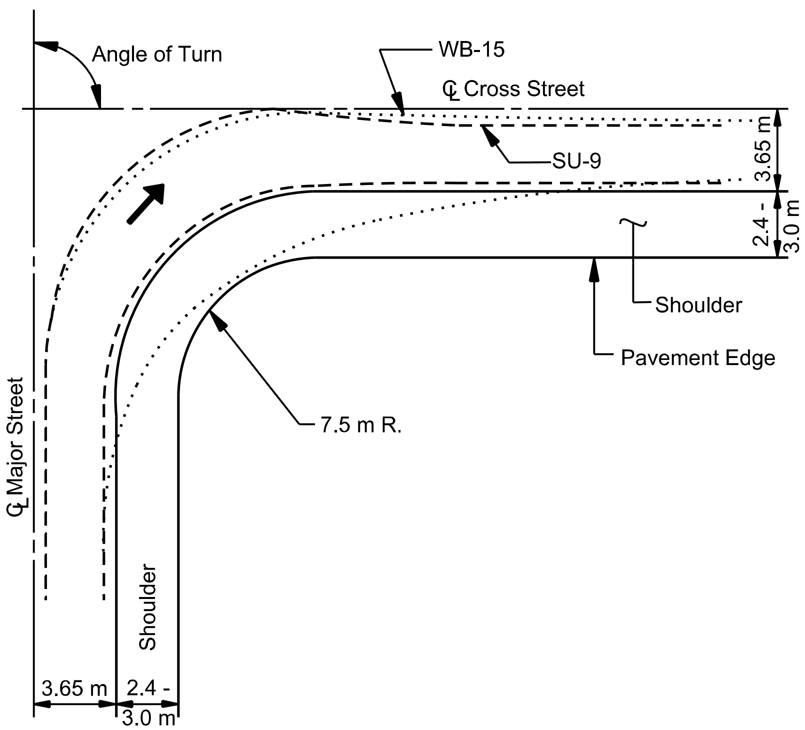
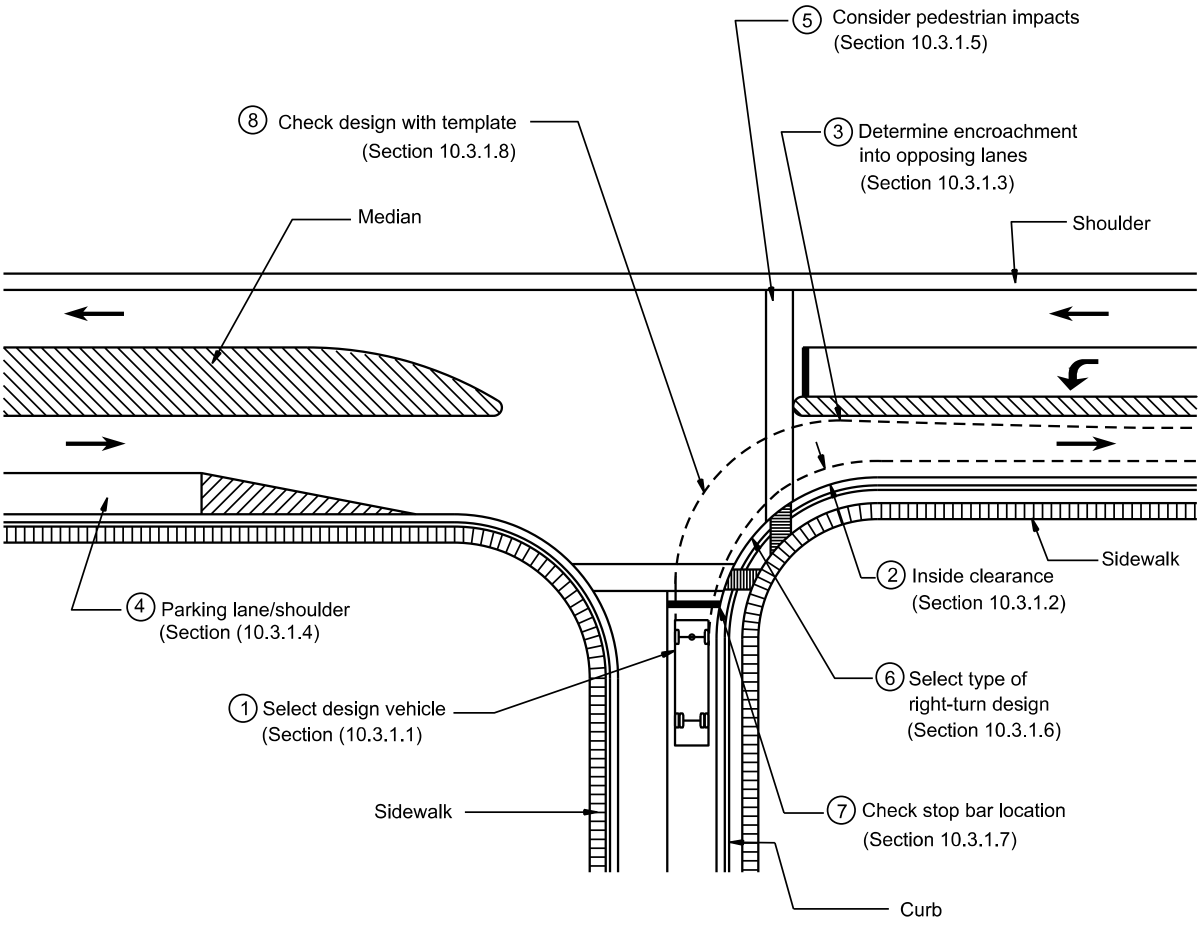


Figure 10-19: Summary of Right-Turn Design Issues

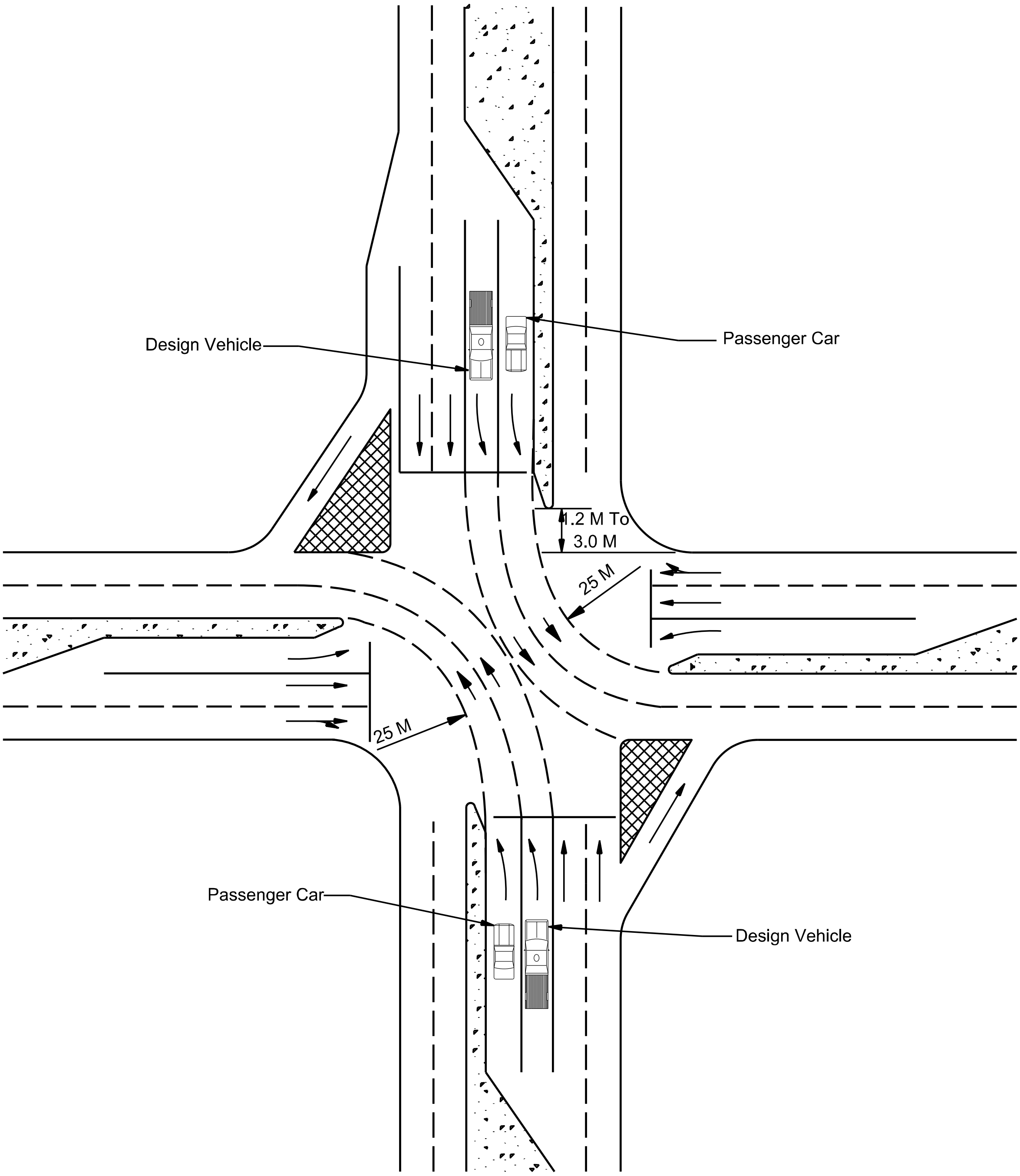


10.3.2 Left-Turn Control Radii

For left turns, the motorist generally has a guide at the beginning and end of the turn and an open intersection in middle. Therefore, simple curves are typically used for left-turn control radii. Occasionally, a three-centred curve may be desirable to accommodate the off tracking of large vehicles.

The design values for left-turn control radii are usually a function of the design vehicle, angle of intersection, number of lanes, and median widths. For roadways intersecting at approximately 90 degrees, radii of 15 m to 25 m should typically satisfy all controlling factors. If centre divisional islands are present, select control radii so that the nose of each divisional island is no closer than 1.2 m or greater than 3.0 m from the edge of the travelled way of the intersecting roadway. See Figure 10-20.

Figure 10-20: Left-Turn Control Radii



10.4 Rural Turning Roadways/Urban Slip Lanes

Where the inner edges of pavements for right turns at intersections are designed to accommodate tractor/semi-trailer combinations or where the desired design permits passenger vehicles to turn at speeds of 25 km/h or greater in rural areas, the pavement area at the corner of the intersection may become excessively large for proper control of traffic. To avoid this, a corner triangle island is used and the connecting roadway between the two intersection legs provided, which is defined as a turning roadway or in urban areas, slip lanes. Figure 10-21 shows an intersection with turning roadways.

Figure 10-21: Intersection with Turning Roadways

10.4.1 Guidelines

The need for a turning roadway will be determined on a case-by-case basis. The designer should consider the following guidelines in determining the need for a turning roadway:

1. <u>Trucks</u>. A turning roadway is usually required when the selected design vehicle is a tractor/semitrailer combination.

2. <u>Island Type and Size</u>. Desirably, the island size should be at least 10 m^2. At a minimum, the island should be at least 9 m^2 in rural areas and 4.5 m^2 in urban areas.

3. <u>Level of Service</u>. A turning roadway can often improve the level of service through the intersection. At signalised intersections, a turning roadway with a free-flow acceleration lane may significantly improve the capacity of the intersection by removing the right-turning vehicles from the signal timing.

4. <u>Crashes</u>. A turning roadway may be considered with a right-turn lane if there are significant numbers of rear-end type crashes at an intersection.

5. Pedestrians. A slip lane will provide a refuge island for pedestrians.

6. Urban. In urban areas, it is desirable to eliminate the slip lane unless it is required for capacity purposes. A traffic analysis of both the intersection turning movements and localised network will be required.

10.4.2 Design

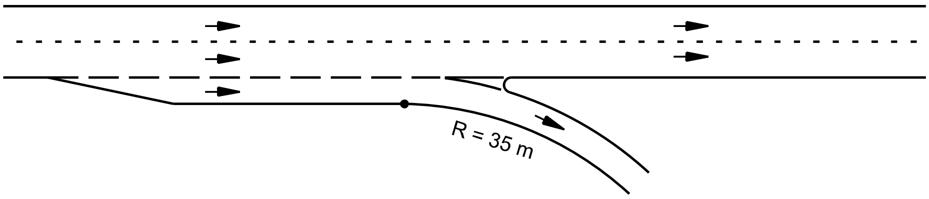
10.4.2.1 Rural Free-Flowing Turning Roadways

Ease and smoothness of operation can result when the free-flow turning roadway is designed with compound curves preceded by a right-turn deceleration lane, as shown in Figure 10-22 and Figure 10-23. The shape and length of these curves should:

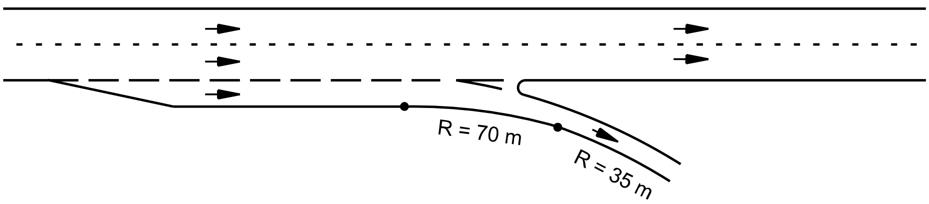
- allow drivers to avoid abrupt deceleration,
- permit development of some superelevation in advance of the maximum curvature, and
- enable vehicles to follow natural turning paths.

The design speed of a free-flow turning roadway for right turns may vary between the end of the right-turn deceleration lane and the central section. The design speed of the turning roadway should be no more than 20 km/h to 30 km/h less than the through roadway design speed.

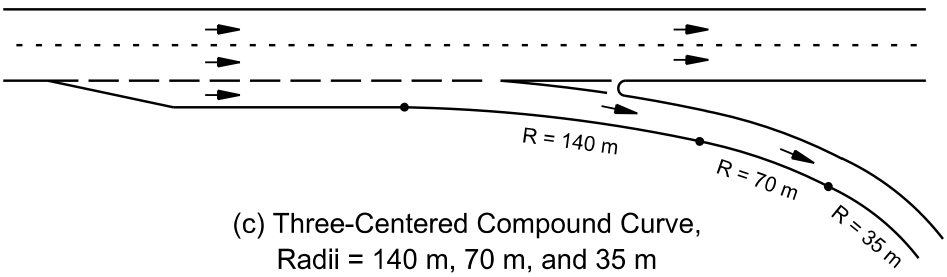
Figure 10-22: Use of Simple and Compound Curves at Free-Flow Turning Roadways



(a) Simple Curve,
Radius = 35 m

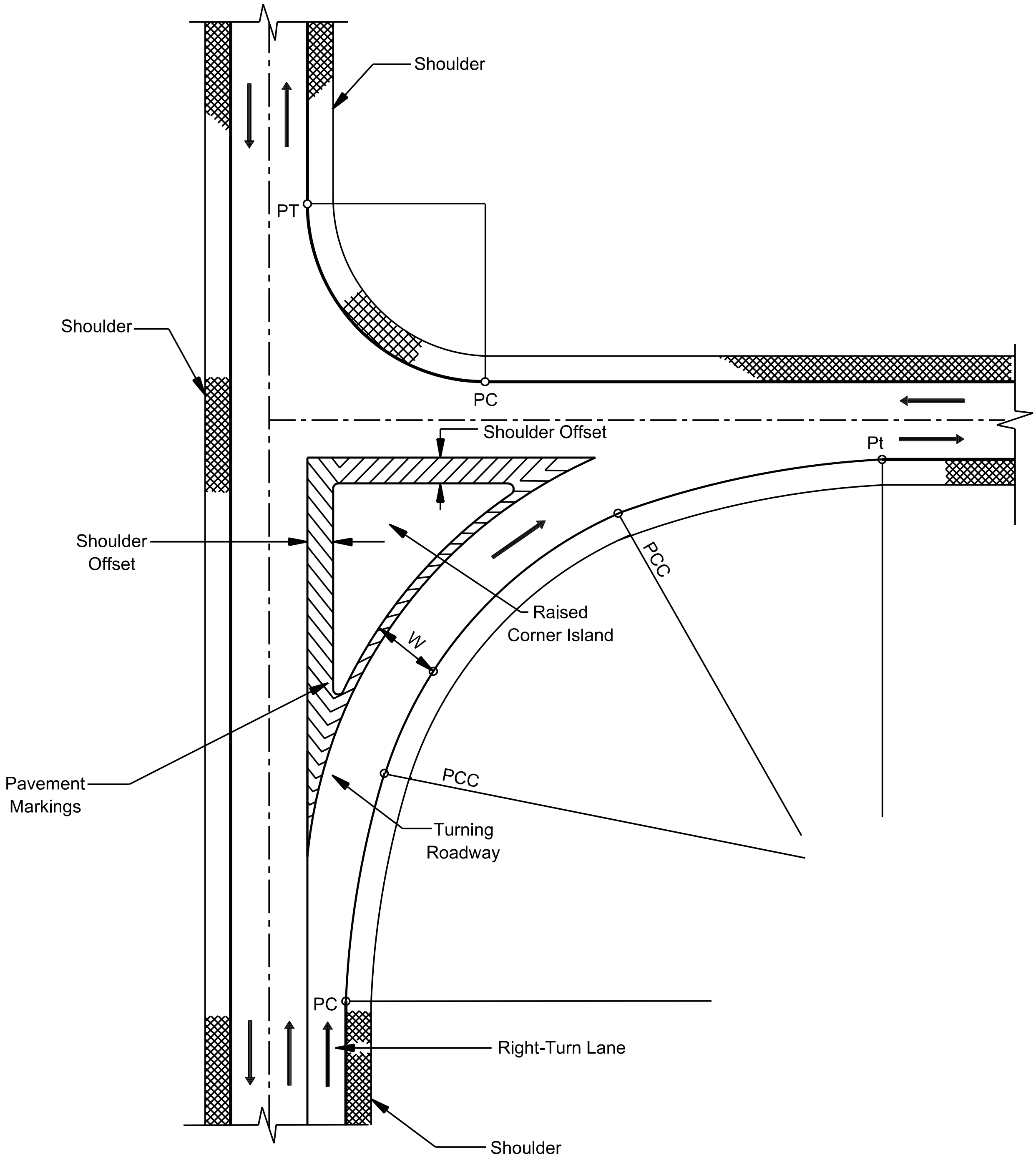


(b) Compound Curve,
Radii = 70 m and 35 m



(c) Three-Centered Compound Curve,
Radii = 140 m, 70 m, and 35 m

Figure 10-23: Typical Rural Turning Roadway Layout



Note: W = width of turning roadway, see Table 10-9*.*

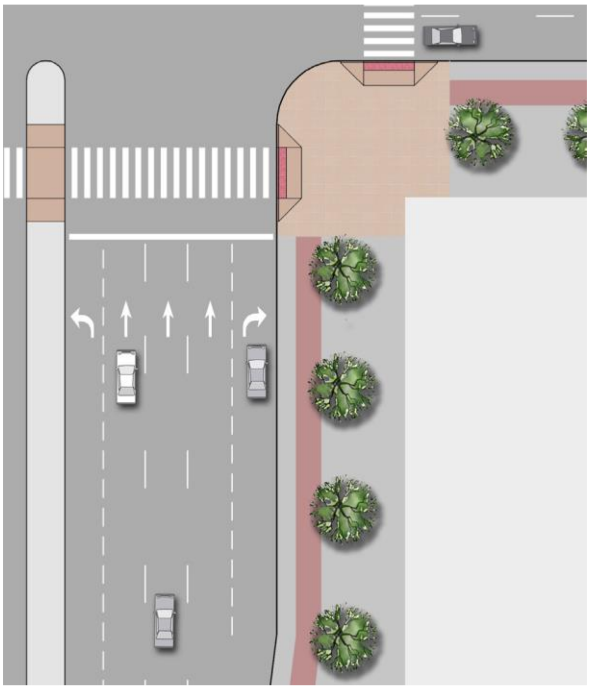
10.4.2.2 Urban Slip Lanes

Figure 10-24 illustrates various options for urban slip lanes. Figure 10-26 provides a typical design for urban slip ramps. The following options for urban slip lanes are as follows in order of preference:

1. Option 1. Figure 10-24(a) shows an intersection without a dedicated right-turn lane and slip lane.

Figure 10-24: Typical Urban Slip Lane Layouts

(a) Option 1 – No slip lane or right-turn lane (preferred option)



(b) Option 2 – No slip lane with dedicated right-turn lane

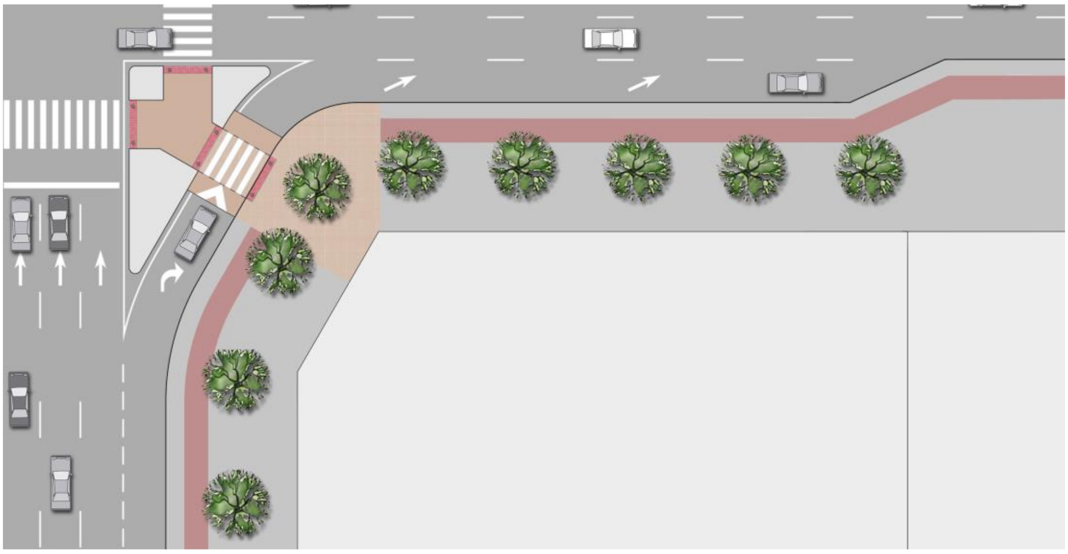
(c) Option 3 – YIELD-controlled slip lane without dedicated right-turn lane

(d) Option 4 – Signalised-controlled slip lane without dedicated right-turn lane)

Source: (4)

Figure 10-25: Typical Urban Slip Lane Layouts

(*Continued*)

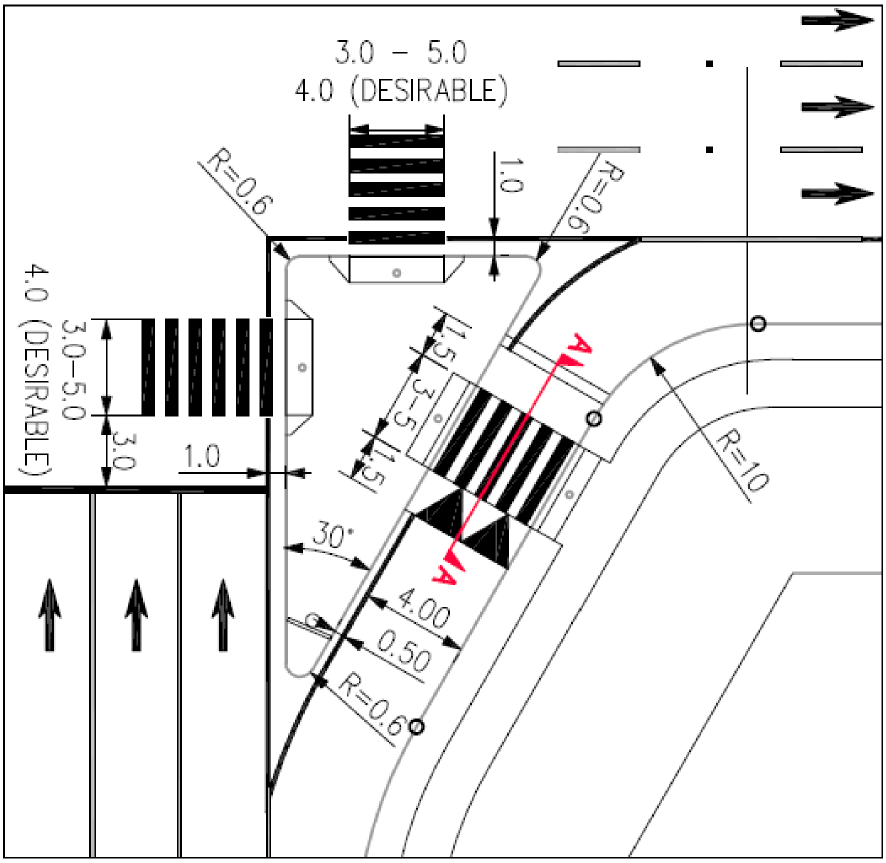


(e) YIELD-controlled slip lane with dedicated right-turn lane and acceleration lane

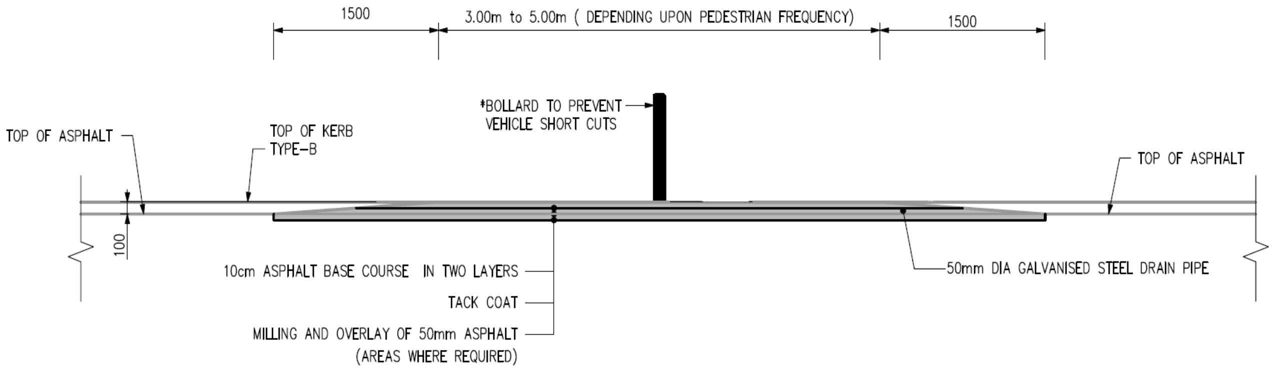
Source: (4)

2. <u>Option 2</u>. Figure 10-24(b) shows an intersection with a dedicated right-turn lane, but without a slip lane.
3. <u>Option 3</u>. Figure 10-24(c) shows a YIELD-controlled right-turn slip lane with a raised pedestrian crossing and no dedicated right-turn lane.
4. <u>Option 4</u>. Figure 10-24(d) shows a signalised right-turn slip ramp without a dedicated right-turn lane.
5. <u>Option 5</u>. Figure 10-24(e) shows a YIELD-controlled slip lane with both a dedicated right-turn lane and an acceleration lane. A traffic analysis should clearly demonstrate the benefits and necessity of acceleration lanes. The auxiliary lane length shall be determined based on the traffic analysis, speed of the traffic on the cross-road, sight distance, and proximity of any sector entrances/exits. In addition to the auxiliary lane length, the taper shall be typically 30 m.

Figure 10-26: Urban Slip Ramp Design



Note: If considerable volume of traffic flow occurs by buses and trucks resulting in large size vehicles being the predominant design vehicles using the right turn slip lane, the width and entry/exit radii of the slip lane maybe increased as per the swept path analysis.



Section A-A*Source:* (4)

10.4.3 Turning Roadway/Slip Lanes Widths

For urban slip lanes, the curb-to-curb width will typically be 4.5 m. Do not use widths greater than 4.5 m as they may encourage drivers to try to use the slip lane as two lanes. The designer should check the design to ensure the selected design and control vehicles are able to make the right-turn without encroaching onto the pedestrian realm.

Rural turning roadway widths are dependent upon the turning radii design, design vehicle selected, angle of turn, design at edges of the turning roadway, and type of operation. Rural turning roadways are designed for one-way operation and are segregated as follows:

1. Case I. One-lane with no provisions for passing a stalled vehicle.
2. Case II. One-lane with provision for passing a stalled vehicle.
3. Case III. Two-lane operation on the travelled way.

Table 10-9 presents guidelines for rural turning roadway widths for various design vehicles based on the above operations. Selection of the appropriate operation will depend on the intersection and will be determined on a case-by-case basis. The following presents several guidelines to consider:

1. Case I. For most rural turning roadway designs, use the Case I widths from Table 10-9. The pavement widths in Table 10-9 provide an extra 1.8 m clearance beyond the design vehicle's swept path. This additional width provides extra room for manoeuvrability and driver variances.

2. Case II and III. Case II and III widths are seldom required on turning roadways. This is due to the relatively short roadway lengths involved. The Case II widths may be appropriate where channelized islands are provided next to through traffic lanes. Case III widths are only applicable where two lanes are used through the turning roadway.

3. Larger Vehicles. In selecting the turning roadway width, the designer should also consider the possibility that a larger vehicle may also use the turning roadway. To some extent, the extra 1.8 m clearances in Case I widths will allow for the accommodation of the occasional larger vehicle at a lower speed and with less clearance. If there are a significant number of the larger vehicles using the turning roadway, it should be selected as the design vehicle.

4. Shoulders. For shoulder designs adjacent to turning roadways, see Figure 10-23.

5. Curbing. Where curb and gutter is provided on the left and/or right side of the turning roadway, add the gutter widths to the widths shown in Table 10-9.

Table 10-9: Design Rural Turning Roadway Widths

Radius on Inner Edge of Pavement, R (m)	Case I, One-Lane, One-Way Operation, No Provision for Passing a Stalled Vehicle (m)							
	P	**SU-9**	**SU-12**	**CITY-BUS**	**WB-12**	**WB-15**	**WB-20**	**WB-33D**
15	4.0	5.5	6.3	6.5	7.0	9.7	-	-
25	3.9	5.0	5.4	5.6	5.8	7.2	9.5*	12.0*
30	3.8	4.9	5.2	5.4	5.5	6.7	8.5*	10.3*
50	3.7	4.6	4.8	5.0	5.0	5.7	6.7	7.7
75	3.7	4.5	4.6	4.8	4.7	5.3	5.9	6.6
100	3.7	4.4	4.5	4.7	4.6	5.3	5.5	6.0
125	3.7	4.4	4.5	4.6	4.5	5.3	5.3	5.7
150	3.7	4.4	4.4	4.6	4.5	5.3	5.2	5.5
Tangent	3.6	4.2	4.2	4.4	4.2	4.4	4.4	4.4
Case II, One-Lane, One-Way Operation With Provision for Passing a Stalled Vehicle by Another of the Same Type (m)								
15	6.0	9.2*	10.9*	11.7*	11.8*	17.3*	-	-
25	5.6	7.9	8.9*	9.5*	9.3*	12.1*	16.8*	21.7*
30	5.5	7.6	8.4*	9.0*	8.8*	11.1*	14.8*	18.4*
50	5.3	7.0	7.5	7.9	7.7	9.1*	11.2*	13.1*
75	5.2	6.7	7.0	7.4	7.1	8.2*	9.6*	10.8*
100	5.2	6.5	6.8	7.1	6.9	7.7	8.8*	9.7*
125	5.1	6.4	6.6	7.0	6.7	7.5	8.3*	9.0*
150	5.1	6.4	6.5	6.9	6.6	7.3	8.0	8.6*
Tangent	5.0	6.1	6.1	6.4	6.1	6.4	6.4	6.4
Case III, Two-Lane, One-Way Operation (same type vehicle in both lanes) (m)								
15	7.8	11.0*	12.7*	13.5*	13.6*	19.1*	-	-
25	7.4	9.7*	10.7*	11.3*	11.1*	13.9*	18.6*	23.5*
30	7.3	9.4*	10.2*	10.8*	10.6*	12.9*	16.6*	20.2*
50	7.1	8.8*	9.3*	9.7*	9.5*	10.9*	13.0*	14.9*
75	7.0	8.5*	8.8*	9.2*	8.9*	10.0*	11.4*	12.6*
100	7.0	8.3*	8.6*	8.9*	8.7*	9.5**	10.6*	11.5*
125	6.9	8.2*	8.4*	8.8*	8.5*	9.3*	10.1*	10.8*
150	6.9	8.2*	8.3*	8.7*	8.4*	9.1*	9.8*	10.4*
Tangent	6.8	7.9	7.9	8.2*	7.9	8.2*	8.2*	8.2*

Note: *Only use the rural turning roadway widths in this table as a guide and check with a turning template or a computer simulated turning template program.*

** Generally, do not provide roadway widths greater than 8.0 m. Instead, consider using larger corner radii.*

Source: (1)

10.4.4 Horizontal Alignment

The horizontal alignment for rural turning roadway design differs from that of open-roadway conditions. In comparison, rural turning roadway designs reflect more restrictive field conditions, and less demanding driver expectation and driver acceptance of design limitations. The following assumptions are used to design the horizontal alignment for rural turning roadways:

1. Curvature Arrangement. The radii designs discussed in Section 10.3.1 (e.g. simple radius with tapers, three centred-curves) are also applicable to turning roadways. For most turning roadway designs, a simple radius with tapers is desirable. A large simple radius can be used where right-of-way is available and where a higher turning speed is desired (e.g. 30 km/h to 40 km/h).

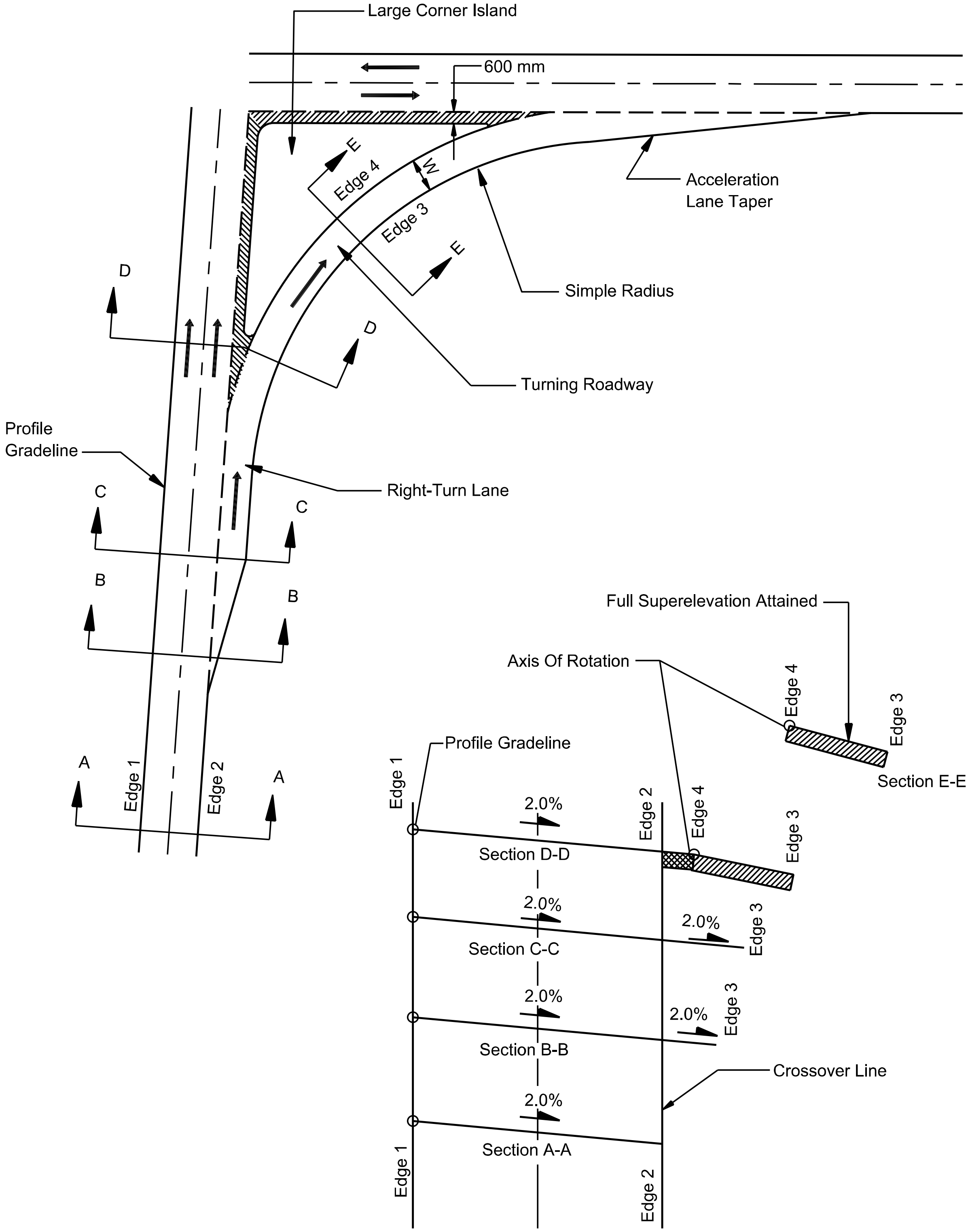
2. Superelevation. Turning roadways are relatively short in length as indicated in Figure 10-23. This increases the difficulty of superelevating the roadway. For turning roadways developed with three-centred curves, a low design speed (e.g. 20 km/h to 30 km/h) is appropriate and the superelevation rate will typically be 2%. The maximum superelevation rate for turning roadways should not exceed 4%. This would apply only where a large simple radius is used. The factors that control the amount of superelevation are the need to meet pavement elevations of the two intersecting roadways, providing for drainage within the turning roadway, and design speed. Selection of the appropriate superelevation rate will be based on field conditions and will be determined on a case-by-case basis.

   Urban slip lanes should not be superelevated. However, provide a 2% cross slope towards the outside away from the island to facilitate drainage.

3. Superelevation Development. Figure 10-27 illustrates a schematic of superelevation development for a turning roadway adjacent to a tangent section of roadway and includes both a right-turn lane and an acceleration-lane taper. The actual development will depend upon the practical field conditions combined with a reasonable consideration of the theory behind horizontal curvature. The following criteria should be met:

   - No change in the normal cross slope is necessary up to Section B-B. Here, the width of the right-turn lane is less than 1 m.
   - At Section C-C, the full width of the right-turn lane is obtained and should be sloped at 2.0%.
   - The full width of the turning roadway should be attained at Section D-D. The amount of superelevation at D-D will depend upon the practical field conditions.
   - Beyond Section D-D, rotate the turning roadway pavement as needed to provide the required superelevation for the design speed of the turning roadway.
   - The superelevation treatment for the exiting portion of the turning roadway should be similar to that described for the entering portion. However, the superelevation rate on the turning roadway at the beginning of the acceleration taper should match the cross slope of the merging roadway.

Figure 10-27: Superelevation Development of Rural Turning Roadway (Mainline on Tangent or Curved to the Right)



Source: (1)

Figure 10-28 illustrates a situation where the mainline curves to the left and away from the crossroad. The designer should make every effort to avoid designing intersections on a curve where superelevation is needed. If this is not practical, the designer can compensate for this problem by proposing the use of a parallel right-turn deceleration lane prior to the turning roadway as shown in Figure 10-28.

4. Cross Slope Rollover. Table 10-10 presents the maximum allowable algebraic difference in the cross slopes between the mainline and the right-turn lane that precedes the turning roadway. In Figure 10-27 and Figure 10-28, this criteria applies between Section A-A and Section D-D. This likely will be a factor only for a superelevated mainline to the left.

5. Minimum Radius. The minimum turning roadway radii are based on the design speed, side-friction factors, and superelevation rate. Table 10-11 presents minimum radii for various turning roadway conditions.

Table 10-10: Maximum Pavement Cross Slopes at Rural Turning Roadways

Design Speed of Turning Roadway Curve (km/h)	Rollover (Algebraic Difference) in Cross Slope at Crossover Line (%)	
	Desirable Maximum	Maximum
20-30	5	8
40-50	5	6
>50	4	5

Note: Values apply between the travelled way and the right-turn lane for rural turning roadways.

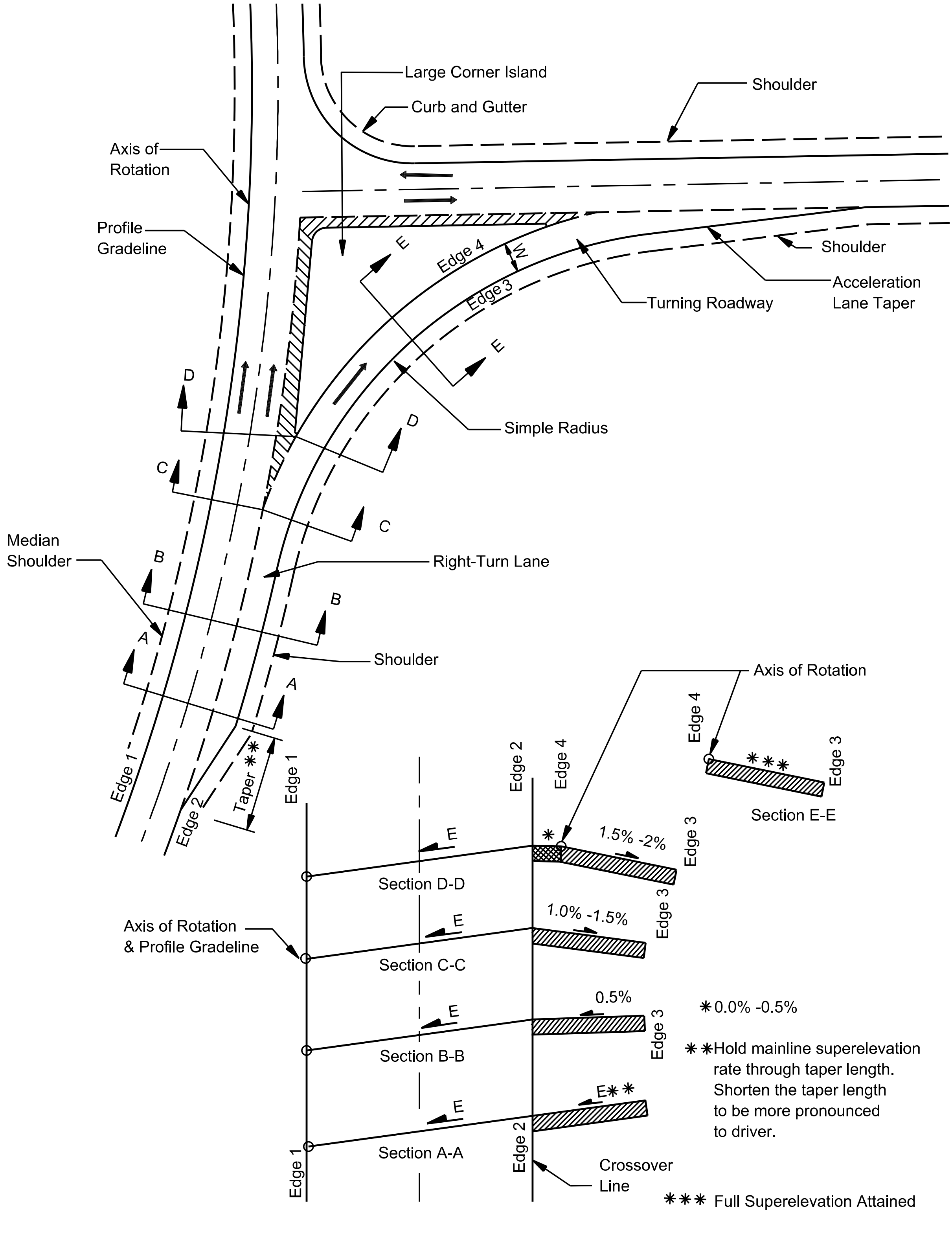
Source: (1)

Table 10-11: Minimum Radii for Rural Turning Roadways

Turning Roadway Design Speed (km/h)	Assumed Maximum Comfortable Side Friction (f)	Assumed Superelevation (e)	Calculated Radius (m)	Design Radius (R_1) (m)
20	0.35	2% 3% 4%	8.5 8.3 8.1	9 8 8
30	0.28	2% 3% 4%	23.6 22.9 22.1	24 23 22
40	0.23	2% 3% 4%	50.4 48.5 46.7	50 49 47
50	0.19	2% 3% 4%	93.7 89.5 85.6	94 90 86
60	0.17	2% 3% 4%	149.2 141.7 135.0	149 142 135

Notes: For design speeds greater than 60 km/h, use open-roadway conditions.

Figure 10-28: Superelevation Development of Rural Turning Road (Mainline Curved to the Left)



Source: (1)

10.4.5 Right-Angle Turns with Corner Islands

The principal controls for the design of rural turning roadways and urban slip lanes are the alignment of the travelled way edge and the turning roadway/slip lane width. These design features ensure that a vehicle can be accommodated while turning at the selected turning design speed. With radii greater than the minimum, the edge of the travelled way controls resulting in an area large enough for an island to be designed between the left edge of the turning roadway/slip lane and the travelled way edges of the two through roadways. Such an island is desirable for delineating the path of through and turning traffic, for the placement of signs, and for providing a refuge for pedestrians and cyclists. Larger islands may be needed to locate signs or to accommodate significant number of pedestrians.

A rural turning roadway/urban slip lane should be designed to provide at least the minimum size island and the minimum width. The roadway width should be wide enough to permit the right and left wheel tracks of a selected vehicle to be within the edges of the travelled way by about 0.6 m on each side.

10.4.6 Oblique-Angle Turns with Corner Islands

The minimum design dimensions for oblique-angle turns are determined on a basis similar to that for right-angle turns, and values are given in Table 10-12. Curve design for the inner edge of the travelled way, the rural turning roadway width, and the approximate island size are indicated for the three chosen design classifications described at the bottom of the table. For a particular intersection, the designer may choose from the three minimum designs shown in accordance with vehicle size, the volume of traffic anticipated, and the physical controls at the site.

In Table 10-12, no design values are given for angles of turn less than 75 degrees. If practical, do not use angles of intersections less than 75 degrees. For flat angles of turn, the design of turning roadways involve relatively large radii and are not considered in the minimum class. Such turning angles should be designed on a case-by-case basis to fit site controls and traffic conditions.

For angles of turn between 75 and 120 degrees, the designs are governed by a minimum island size, which provides for larger turns than minimum turning radii. For angles of turn 120 degrees or more, the sharpest turning path of a design vehicle is selected and the curves on the inner edge of travelled way generally control the design, which results in an island size greater than the minimum. In Table 10-12, the inner edge of travelled way arrangement for designs B and C for turning angles between 120 and 150 degrees are the same for single-unit trucks and semitrailer combinations, respectively. The size of islands for the larger turning angles given in the last column of Table 10-12 indicates the areas of unused pavement that are eliminated by the use of islands.

Table 10-12: Typical Designs for Rural Turning Roadways

Angle of Turn (degrees)	Design Classification	Three-Centred Compound Curve		Width of Lane (m)	Approx. Island Size (m^2)
		Radii (m)	Offset (m)		
75	A	45-23-45	1.0	4.2	5.5
	B	45-23-45	1.5	5.4	5.0
	C	67-41-67	1.5	6.7	33.5
90	A	45-15-45	1.0	4.2	5.0
	B	45-15-45	3.4	6.4	14.0
	C	61-21-61	3.4	7.6	25.0
105	A	36-12-36	0.6	4.5	6.5
	B	46-11-46	3.5	8.8	6.0
	C	55-18-55	2.9	9.8	24.0
120	A	30-9-30	0.8	4.8	11.0
	B	46-9-46	3.2	10.0	12.0
	C	43-17-43	2.1	13.7	20.0
135	A	30-9-30	0.8	4.8	43.0
	B	46-9-46	3.0	11.6	37.0
	C	43-14-43	2.1	15.8	45.0
150	A	30-9-30	0.8	4.8	130.0
	B	46-9-46	2.7	12.8	125.0
	C	49-12-49	1.8	16.1	150.0

Notes: Asymmetric three-centred compound curve and straight tapers with a simple curve can also be used without significantly altering the width of roadway or corner island size. Painted island delineation is recommended for islands less than 7 m^2 in size.

Design classification:

A — Primarily passenger vehicles; permits occasional design single-unit trucks to turn with restricted clearances

B — Provides adequately for the SU design vehicles; permits occasional WB-20 design vehicles to turn with slight encroachment on adjacent

C — Provides fully for the WB-20 design vehicle.

Source: (1)

10.4.7 Deceleration/Acceleration Lanes

Deceleration and acceleration lanes should not be used in urban areas except for major arterials. Figure 10-29 shows a turning roadway with both deceleration and acceleration lanes. Consider the following guidelines for using an acceleration or deceleration lanes:

1. Urban Slip Lanes. Desirably, deceleration/acceleration lanes should not be used with urban slip lanes.

2. Deceleration Lane Guidelines. Consider the following guidelines for including a deceleration lane prior to a rural turning roadway:

a. Turning Roadway Design Speed. A right-turn deceleration lane may be considered where the turning-roadway design speed is more than 30 km/h lower than that of the mainline design speed.

b. Storage Length. A right-turn deceleration lane may be beneficial at signalised intersections where the through lane storage may limit access to the rural turning roadway. In these cases, the deceleration lane should extend upstream beyond the through lane queue requirements.

3. Acceleration Lane Guidelines. Consider the following guidelines for including an acceleration lane after the turning roadway:

   a. Traffic Condition. Consider providing an acceleration lane where it is desirable to provide a free-flowing traffic merge. The acceleration lane should not be preceded by a stop or yield condition.

   b. Traffic Volumes. Consider providing an acceleration lane where the turning traffic must merge with the through traffic of a high-speed, high-volume facility and/or where there is a high volume of trucks turning onto the mainline.

   c. Sight Distance. Acceleration lanes may be considered if there is inadequate sight distance available to allow the driver to safely merge with the mainline facility.

4. Deceleration Lane Design. For guidance on the design of right-turn deceleration lanes, see Section 10.7.2.

5. Acceleration Lane Design. Design acceleration lanes at intersections in the same manner as for interchange ramps using the parallel design. See Section 12.6.2. However, those distances for interchanges may be excessive under restricted urban conditions, where the length of acceleration lanes for urban intersections may be suitably adjusted. Taper lengths may vary from 30 m to 45 m instead of 90 m. The below values are recommended where acceleration lanes are required at Signalized Intersections and where right-in right-out (RIRO) located on urban expressways and arterial roads. Traffic studies and Road Safety audit shall be undertaken for selecting the proper acceleration length as per the site constraints.

Design Speed (kph)	Painted Nose for RIRO's Length (m)	Acceleration Lane Length (m) (Excluding Taper & Nose)	Taper Length (m)
80	-	40 to 90	30
100	20	80 to 110	40
120	40	100 to 130	45

Recommended Acceleration Lane Lengths in Urban areas

10.5 Channelization

Channelization is the separation or regulation of conflicting traffic movements into definite paths of travel by traffic islands or pavement markings to facilitate the orderly movements of both vehicles and pedestrians. Proper channelization increases capacity and provides positive guidance to motorists; improper channelization has the opposite effect and may be worse than none at all. Too much channelization should be avoided because it could create confusion and worsen operations. A simple channelization improvement can sometimes result in dramatic operational efficiencies and reduction in crash frequencies. Figure 10-29 illustrates a channelized intersection.

10.5.1 Guidance

The following lists some of the advantages for providing channelization at intersections:

- Separation of left-turn movements from through traffic reduce rear-end crashes and provide a comfortable means for making a left turn.
- Paths of vehicles can be confined so that no more than two paths cross at any one point.
- The angle and location at which vehicles merge, diverge, or cross can be controlled.

Figure 10-29: Channelized Intersection

- The amount of pavement for vehicles is reduced and thereby decreases the potential for vehicles to wander and narrows the area of conflict between vehicles.
- Clearer indications are provided for the proper path movements.
- The predominant movements can be given priority.
- Areas are provided for pedestrian refuge.
- Space is provided for traffic control devices so that they can be more readily perceived.
- Prohibited turns are controlled.
- The vehicular speeds can be restricted.
- Traffic movements can be separated where multiple phased traffic signals are used.
- Wrong-way entry of freeway ramps, one-way streets, and turning roadways may be discouraged. Signs and supplementary pavement markings are among the most important devices to discourage wrong-way turns.

10.5.2 Design Considerations

The following elements control the design of the channelized intersection:

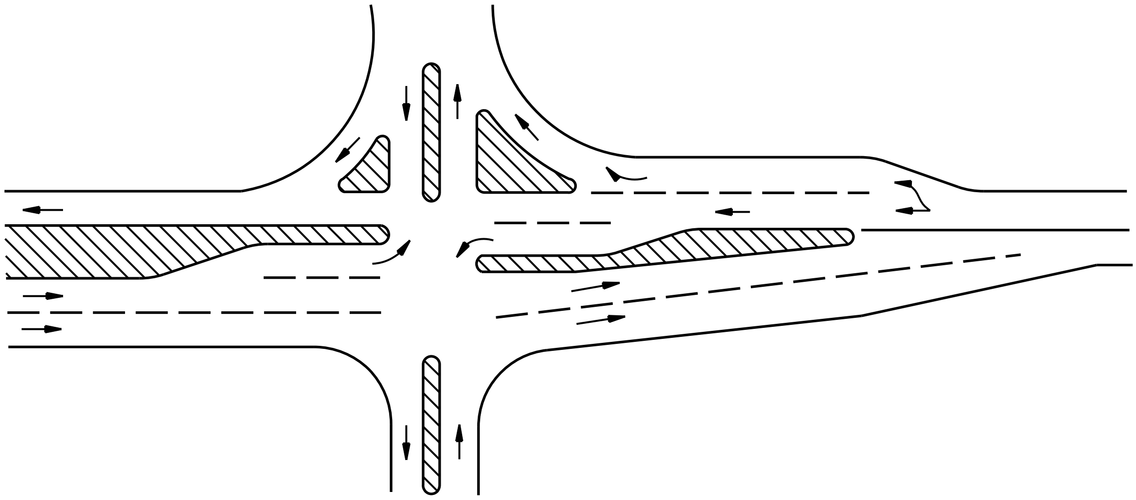
- design vehicle,
- cross sections on the crossroads,
- projected traffic volumes in relation to capacity,
- number of pedestrians and cyclists,
- speed of vehicles,

- location of any needed bus stops,
- type and location of traffic control devices,
- right-of-way, and
- terrain.

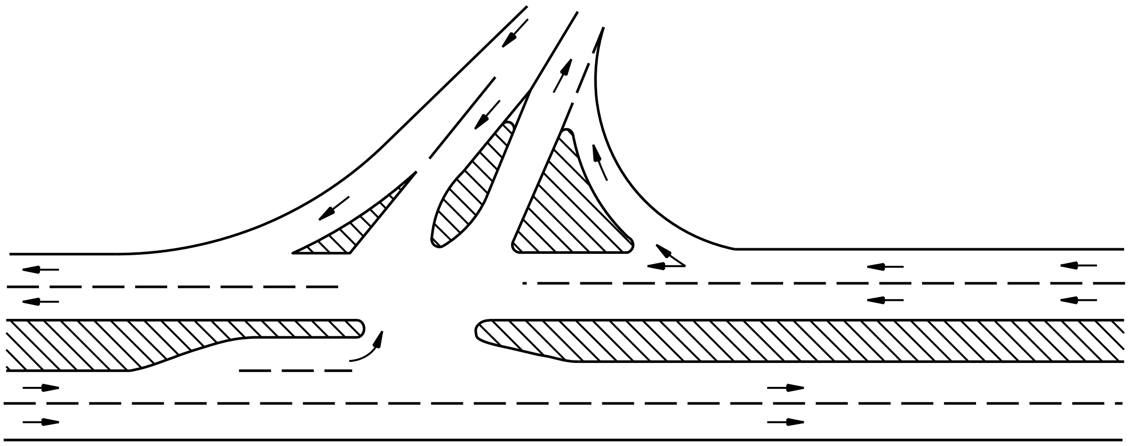
The designer should consider the following guidelines in the design of channelized intersections:

- Motorists should not be confronted with more than one decision at a time.
- Desirable or safe vehicular speeds should be encouraged. Channelization should promote desirable vehicle speeds wherever practical. In other cases, channelization may be used to limit vehicle speeds in order to mitigate serious high-speed conflicts.
- Avoid unnatural paths that involve turns greater than 90 degrees or sudden and sharp reverse curves.
- Reduce the number conflict points as much as practical.
- Keep vehicles within well-defined paths that minimise the area of conflict. Desirable vehicular paths should be clearly defined; see Figure 10-30.
- The operating characteristics and appearance of intersections should reflect and facilitate the intended high priority traffic movements. Selection of high priority movements can be based on relative traffic volumes, functional classification of the intersecting highways, or route designations.
- Provide long merging and weaving areas as conditions permit.
- Where the distance to a downstream driveway or intersection is less than the desirable distance for merging or weaving and where pedestrians are present, provided a YIELD sign, STOP sign, or traffic signal on the turning roadway.
- Non-weaving or non-merging traffic streams should intersect at angles close to 90 degrees as practical, with a range of 60 to 120 degrees considered acceptable. When traffic streams cross without traffic signal control, the crossing should be made at or near right angles in order to reduce the potential impact areas, to reduce the time of crossing a conflicting traffic stream, and to provide the most favourable sight lines for drivers to judge relative positions and relative speeds of other vehicles.
- Traffic should merge at small angles. Merging at angles of 10 to 15 degrees permits traffic streams to flow together with minimum speed differentials. Drivers entering the major traffic flow may only require relatively short gaps.
- Decelerating, stopped, or slow vehicles should be removed from high-speed through-traffic streams; see Figure 10-30(c). Wherever possible, intersection designs should produce separation between traffic streams with large traffic speed differentials.

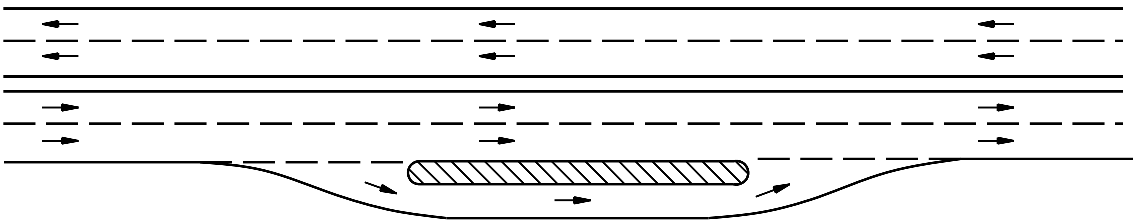
Figure 10-30: Examples of Channelized Intersections



(a)



(b)



(c)

- Ensure adequate sight distance can be provided for merging streams of traffic. Note that the angle of the intersection will affect the motorist's line of sight.
- Study all points of crossing or conflict to determine if such conditions would be better separated or consolidated to simplify the design.
- Provide turning vehicles a refuge area to separate the turning vehicle from the through traffic. Clearly delineate turning lanes to encourage their use by turning drivers; see Figure 10-30(a) and Figure 10-30(b).
- Ensure islands used for channelization do not interfere with or obstruct cycle lanes at intersections.
- Islands should not cause confusion about the proper direction of travel around them.
- Prohibited turns should be blocked by channelizing islands, wherever practical, to restrict or prevent wrong way movements. Where such movements cannot be completely blocked, design the channelization scheme to discourage their turns; see Figure 10-30(b).
- Provide safe refuges for pedestrians and other non-motor vehicle users. Channelization can shield or protect pedestrians, cyclists, and the disabled within the intersection area. Proper use of channelization will minimise exposure of these vulnerable users to vehicle conflicts, without hindering vehicular movements.
- Location of traffic islands, medians, and curb returns should reflect consideration of the need to place signals and signs in locations visible to drivers. These devices are discussed in the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

10.6 Islands

Several of the treatments described in this chapter require channelizing islands within the intersection area. Some intersections, especially those with oblique angle crossings, result in large paved areas that may cause motorists to wander from natural or expected paths and may cause long pedestrian crossings. These movements may result in conflicts and/or unpredictable operations, but could be enhanced by incorporating channelizing islands in the design of the intersection. Figure 10-31 shows an example of a channelized island within an intersection.

At rural locations where higher speeds are prevalent, channelizing islands are used in conjunction with left-turn lanes and for turning roadways. In urban areas where speeds generally are lower, but where traffic volumes are generally higher, channelizing islands in conjunction with added lanes are used primarily to increase capacity and safety at the intersection.

Figure 10-31: Channelize Island

10.6.1 Island Types

Islands can be grouped into the following classifications. Most island types serve at least two of these functions:

1. <u>Corner/Directional Islands</u>. Directional or corner triangular islands control and direct right-turn movements and guide the driver into the proper direction.

2. <u>Channelizing Islands</u>. Centre channelizing islands separate opposing traffic flows, alert the driver to the crossroad ahead, and regulate traffic through an intersection. These islands are often introduced at intersections on undivided roadways and are particularly advantageous in controlling left turns at skewed intersections.

3. <u>Refuge Islands</u>. Refuge islands (corner islands or centre channelizing islands) may function to aid and protect pedestrians who cross a wide roadway. These islands may be required for pedestrians where complex traffic signal phasing is used and may permit the use of two-stage crossings. This also may increase the signal efficiency by allowing the time allocated for pedestrian movements to be reduced. The refuge Island should be adequately sized for pedestrian and cyclists storage.

10.6.2 Selection of Island Type

Islands may be some combination of flush, raised, or turf, and could be triangular or elongated in shape. Selection of an appropriate type of channelizing island should be based on:

- traffic characteristics,
- cost considerations,
- urban or rural locations,
- degree of access management desired, and
- maintenance considerations.

The following sections offer guidance where different types of islands are appropriate.

10.6.2.1 Flush Islands

Flush islands, which are delineated by pavement markings (e.g. paint, thermoplastic, epoxy) are appropriate:

- on roadways to delineate separate left-turn lanes;
- in restricted locations where delineation of vehicular path is desirable, but space for larger, raised islands is not available;
- to separate opposing traffic streams on low-speed urban streets; and/or
- for temporary channelization during construction
- the use of flush islands for separation of opposing traffic streams should be avoided

10.6.2.2 Raised Islands

Raise islands, with curbs that are at least 100 mm high, are appropriate:

- on low-speed roadways where the primary function is to provide positive separation for opposing traffic movements;
- at locations requiring positive delineation of vehicular paths (e.g. where a major route turns or at intersections with unusual geometry);
- where the island is intended to prohibit or prevent traffic movements (e.g. wrong-way movements or to manage access within the intersection);
- where a primary or secondary island function is to provide a location for traffic signals, signs, or other fixed objects; and/or
- where a primary function of the island is to provide a pedestrian refuge.

Raised islands should only be used at rural intersections having the following characteristics:

- on the crossroad through an interchange to delineate median crossovers and turn lanes, and to prevent wrong-way movements, and
- at unusual or complex intersection configurations where higher visibility would promote greater safety and more efficient traffic operations.

Where curbing is proposed in high-speed rural areas, only use sloping curbs and consider providing supplemental intersection illumination. In addition, provide prismatic reflectors on the top of curbs to enhance delineation of the island and turn lanes at night.

10.6.2.3 Pavement Edge Islands

Channelizing islands formed by pavement edges generally only apply to rural areas. One example of this channelization type is where a divided four-lane facility with a median ditch section is temporarily tapered to a two-lane roadway section. This reduction of the four lanes down to two is considered channelization.

10.6.3 Design of Islands

10.6.3.1 Channelizing Islands

Because centre channelizing islands (flush or raised) are often introduced within the travelled way, special care is necessary in their design to ensure that they do not become a hazard. The designer should consider the following criteria:

1. Nose. Place the noses of raised-curb islands so that they are conspicuous to approaching motorists and are outside of the assumed vehicular path. This clearance should be both physical and visual so that drivers will not veer away from the island.

2. Nose Ramping. Consider ramping the approach nose of curbs where:

    - a raised median or curbed centreline channelization is introduced to separate opposing lanes of traffic,

    - a change is made from a flush to a raised median, and

    - median crossovers or openings are outlined with curbing.

    At locations that are designed for the protection of pedestrians, traffic signals, light standards, or sign supports, nose ramping should not be considered.

3. Alignment. Provide a smooth, free-flowing alignment both into and out of a dual carriageway. On entering the channelized approach, widen the travelled way out opposite the curbed nose and gradually transition it to the normal divided travelled way width. In addition, provide a gradual transition on the departure side of the dual carriageway. Where two lanes are being funnelled down to one lane on the departure side of the channelizing island, provide sufficient pavement width and/or an outside paved shoulder at the curbed nose to provide some lateral escape clearance for merging vehicles. This is important where a motorist has failed to observe the single-lane warning signs and is still operating two abreast as the vehicle approaches the transition to two-lane, two-way operations.

4. Island Size. Traffic channelizing islands should be large enough to command the driver's attention. Island shapes and sizes vary from one intersection to another. The minimum width for a flush-type island on new construction is 3.0 m and for reconstruction projects 1.5 m.

5. Island Length. The island should be of sufficient length to forewarn a motorist of an approaching intersection and to provide space for the proper development of a free-flowing alignment. The edge of the travelled way, the width of the divided roadways, and the width of the centre channelizing island normally control the length of island and the pavement edge radii. As a guide, 3.0 seconds at the design speed may be used.

6. Delineation. Channelizing islands should be delineated based on their size, location, and function. Raised islands present the most positive means of delineation. Where space is limited, use paint to delineate the island. Raised pavement markers, curb-top reflectors, or paint striping can be used in advance of and around an island to help alert the driver of an approaching island. These traffic control devices are especially important at the approach to raised divisional islands.

Round the approach and merging ends of curbed islands according to Figure 10-32.

7. Offsets. Figure 10-32 provides guidance on the applicable offsets that should be used with curbed-channelizing islands.

8. Corrugated Median Surface. In advance of the curbed nose of a divisional island, consider providing a sufficient length of corrugated median that allows the driver enough warning time to move away from the raised island. As a guide, 1.5 seconds of travel time at the design may be used.

9. Cross Slopes. With centre curbed-channelizing islands, up to approximately 10 m wide and where such islands are located on tangent segments or on very flat curvature, the length of the island normally provides sufficient distance for gradual lateral shifts of traffic either to the right (entering) or to the left (departing). Because the required lateral shifts usually are not greater than the normal rate of lane shifts made during a passing measure, the cross slope of the pavement through the channelized approach can be unidirectional at 2% sloped away from the island.

10. Stopping Sight Distance. At a minimum, provide stopping sight distance to the ramped nose of the island. Desirably, provide decision sight distance to the ramped nose.

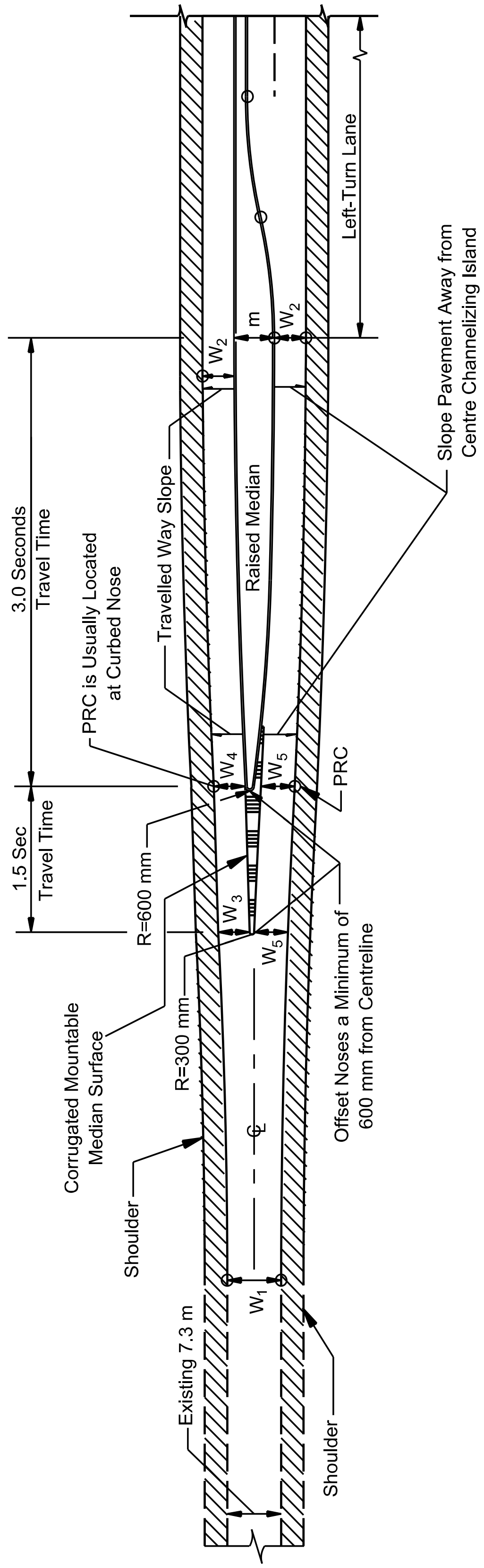
11. Typical Designs. Figure 10-32 illustrates a typical curbed divisional island and applicable approach treatment.

12. Simplicity. Do not introduce divisional islands in areas that can create confusion due to complexity or which cause excessive restrictions. Complex intersections that present multiple choices of movement are undesirable. Ensure that the design remains simple to eliminate possible confusion.

10.6.3.2 Curb Ramps

If the crosswalk is placed through an island, give special consideration to the treatment of curb ramps within the raised island. In many cases, the crosswalk can be located directly in front of a divisional island nose without special design provisions or the island can be shortened sufficiently to permit such location without loss of control for turning vehicles; see Figure 10-19. However, where an island does encroach on the location of a crosswalk, it is usually desirable to depress the entire crosswalk through the island, rather than construct ramps; see Figure 10-33. This is particularly true if the island is less than 10 m^2 or is less than 5.0 m wide. The remaining portion of raised island on either side of the ramp should be of sufficient size to distinguish it as a raised island and for ease of construction.

Figure 10-32: Typical Channelizing Island Design



Note: The length and shape of channelizing islands derived from the above sketch also may be used as a guide for determining a flush, centre island design.

M = typical median widths
W_1 = Undivided approach width
W_2 = Divided approach width
W_3 = $\frac{W_1}{2}$
W_4 = $\frac{W_3 + W_2}{2}$, desirable
W_5 = W_2 + 300 mm
PRC = Point of Reverse Curves

Figure 10-33: Crosswalk Through Centre Island

Source: (4)

10.6.4 Refuge Islands

A refuge island for pedestrians is one at or near a crosswalk or cycle path that aids and protects pedestrians and cyclists who cross the roadway. Raised corner islands and centre channelizing or divisional islands can be used as refuge areas. Refuge islands for pedestrians and cyclists crossing a wide street, for loading or unloading transit riders, or for curb ramps are used primarily in urban areas. The designer should note the following:

- Refuge islands should be located within clearly visible areas.
- The maximum uninterrupted crossing distance without a median refuge should not exceed four travel lanes.
- The location and width of crosswalks, the location and size of transit loading zones, and the provision of curb ramps influence the size and location of refuge islands. Refuge islands should be a minimum of 2.0 m wide. In addition, they should be wide enough to store anticipated volumes of pedestrians, strollers, and cyclists (typically 10 m^2; 12 m^2 on Boulevards).
- Pedestrians and cyclists should have a clear path through the island and should not be obstructed by poles, sign posts, utility boxes, etc.
- Pedestrian crossings may be staggered or angled to orient pedestrians to oncoming traffic.

- Provide tactile paving to warn visually impaired people when they enter the travelled way.
- Consider using shade structures when trees cannot be used to shade crossings at medians.

In both rural and urban areas, many of the islands designed for the function of channelization also serve as refuge for pedestrians. The general principles for island design also apply directly to providing refuge islands.

10.6.5 Corner Islands

It may be desirable to provide a directional or corner island to direct drivers. This may be especially advantageous where a tractor/semi-trailer is used as the design vehicle and/or at oblique angle crossing intersections. The corner island may also be used for locating traffic control devices.

Corner islands may also function as refuge islands to aid and protect pedestrians who cross a wide roadway. Corner islands may be required to pedestrians where complex signal phasing is used, and they may permit the use of two-stage crossings. This may enhance traffic signal efficiency by allowing a reduction in the time allocated for pedestrian movements. The type and size of triangular or corner islands will vary according to the angle of intersection, design vehicle, right-turn operation, and available right-of-way. Figure 10-34 illustrates the typical design for corner islands. In addition, consider the following:

1. <u>Island Sides</u>. The sides of the island should not be less than 3.6 m and, preferably 4.5 m after rounding the corners. This is for non-pedestrain areas and for pedestrian areas the size may increase and will be based on pedestrian flows. If traffic signal posts are installed within the island, the sides of the island should be 9.0 m or greater.

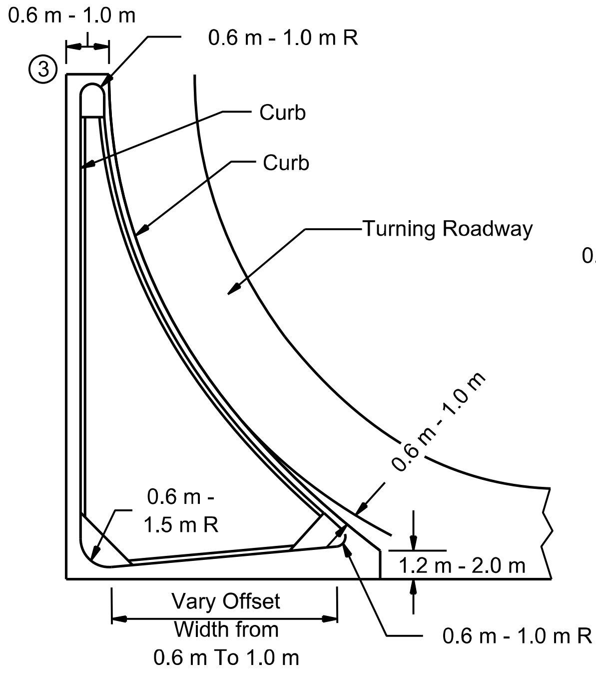
2. <u>Island Size</u>. The minimum island size for rural areas is 9.5 m^2. For urban islands, the island area typically should be 7.0 m^2 but not less than 5.0 m^2 , and this may increase based on pedestrian flows. The island area includes the concrete median surface and the top of the curb.

3. <u>Flush or Raised</u>. For proper delineation of corner islands, under all conditions (e.g. night time, rain, fog), the raised design is preferable.

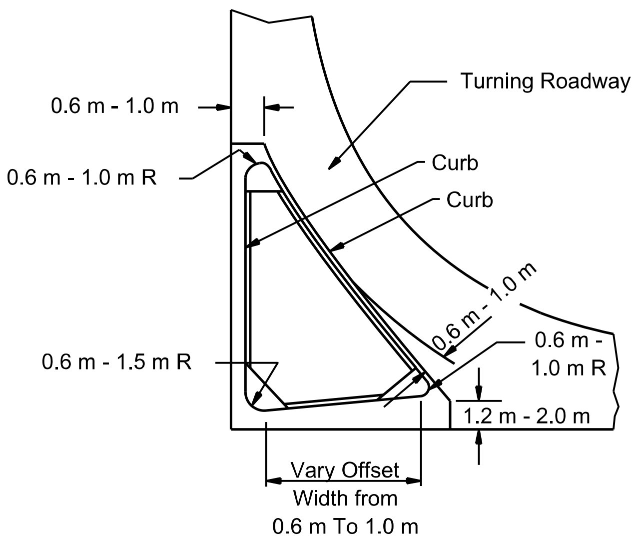
4. <u>Curbing</u>. Consider the following:

    - Use vertical curbs on islands that are located adjacent to a roadway with speeds of 70 km/h or less.
    - Use sloping curbs on rural islands that located adjacent to high-speed traffic (80 km/h or greater). However, use vertical curbs on islands where traffic signal supports, sign truss supports, or any other post with a foundation generally larger than a standard roadway sign are present..
    - Use sloping curb on all sides of rural islands where the island is offset the shoulder width from the edge of the travelled way.

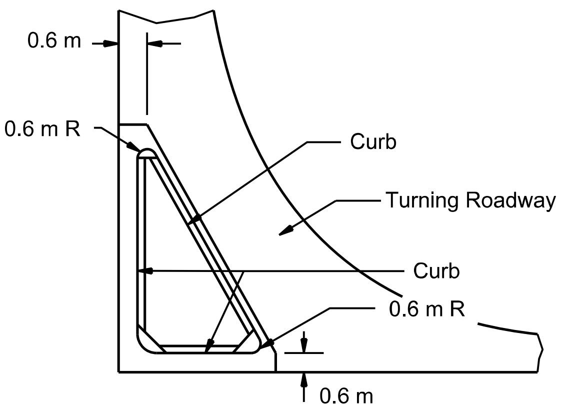
Figure 10-34: Details of Corner Islands



(a) Large Island



(b) Intermediate Island



(c) Small Island

Notes:

1. *①, ②, ③ — designates a specific corner of island.*
2. *Ramp the ① and ② noses of curbed corner islands unless the curb function is for the protection of pedestrians, signals, light standards, or sign truss supports.*
3. *See the Abu Dhabi Design Standards* (27) *for details of ramping noses.*
4. *All corner radii are to the face of curb at the flowline.*
5. **These dimensions are controlled by the minimum area requirements of the island, whereas "W" is a required dimension.*

10.6.6 Island Size and Designation

Island sizes and shapes vary from one intersection to another. Islands should be sufficiently large to command attention. Generally, the smallest curbed corner island normally should have an area of approximately 5 m^2 for urban and 7 m^2 for rural intersections. However, 9 m^2 is preferable for both. Accordingly, corner triangular islands should not be less than about 3.5 m, and preferably 4.5 m, on a side after the rounding of corners.

Elongated or divisional islands should be not less than 1.2 m wide and 6 m to 8 m long. In special cases where space is limited, elongated islands may be reduced based on-site conditions. If used for pedestrian refuges, provide a minimum width of 2.0 m. In general, introducing curbed divisional islands at isolated intersections on high-speed roadways is undesirable unless special attention is directed to providing high visibility for the islands (e.g. painting the curb). Curbed divisional islands introduced at isolated intersections on high-speed roadways should be 30 m or more in length. When situated near a high point in the roadway profile or at or near the beginning of a horizontal curve, the approach end of the curbed island should be extended to be clearly visible to approaching drivers.

Islands should be delineated or outlined by a variety of treatments, depending on their size, location, and function. The type of area in which the intersection is located, rural versus urban, also governs the design. In a physical sense, islands can be divided into three groups:

1. raised islands,
2. islands delineated by pavement markings or reflectorized markers placed on paved areas, and
3. islands formed by the pavement edges and possibly supplemented by delineators on posts or other guideposts, or mounded-earth treatment beyond and adjacent to the pavement edges.

The curb raised island treatment is universal and provides the greatest positive guidance. In rural areas where curbs are uncommon, this treatment often is limited to corner islands of small to intermediate size. Conversely, in urban areas the use of this type of island is common.

Island delineation of unused paved areas, by pavement markings, is common in urban districts where speeds are low and space is limited. In rural areas, this type may be used to minimise maintenance problems on high approach speeds. Group 2 islands also are applicable on low-volume roadways where the added expense of curbs may not be warranted and where the islands are not large enough for delineation by pavement edges alone.

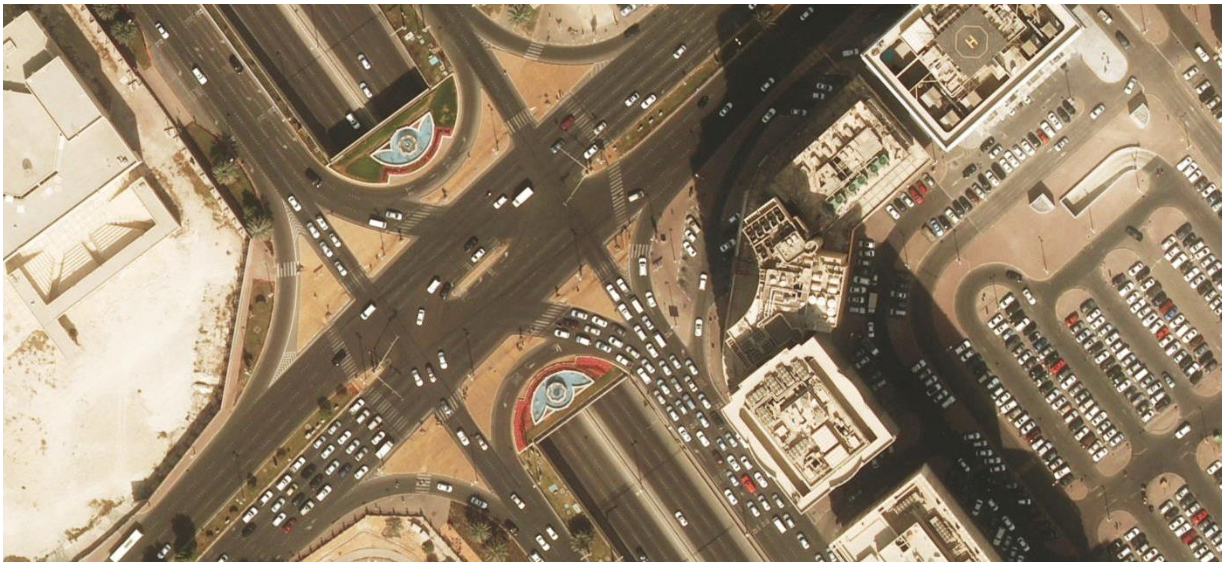
The Group 3 treatment by its nature applies to other than small channelizing islands and is primarily used at rural intersections where there is space for large-radius intersection curves and wide medians.

The central area of large channelizing islands in most cases has a turf or other vegetative cover; see Figure 10-31. As space and the overall character of the roadway determine, low plant material may be included, but it should not obstruct sight distances. Ground cover or plant growth can be used for channelizing islands and provides excellent contrast with the paved areas, assuming that the ground cover is cost-effective and can be properly maintained. Small curbed islands may be mounded, but where pavement cross slopes are outward, large islands should be depressed to avoid draining water across the pavement. For small curbed islands and in areas where growing conditions are not favourable, some type of paved surface is used on the island.

10.7 Auxiliary Turn Lanes

When turning manoeuvres for left- and right-turning vehicles occur from the through travel lanes, it typically disrupts the flow of through traffic. This is especially true on high-volume roadways. To minimise potential conflicts and to improve the level of service and safety, the use of turn lanes may be warranted for intersections. Figure 10-35 illustrates an urban intersection with left- and right-turn lanes.

Figure 10-35: Urban Intersection with Turn Lanes



10.7.1 Warrants

Many factors enter into the choice of type of intersection and the extent of design of a given type, but the principal controls are the design-hour vehicular, pedestrian and cyclist volumes, character or composition of traffic, and design speed. The character of traffic and design speed affects many details of design, but in choosing the type of intersection, they are not as significant as the traffic volumes. Of particular significance are the actual and relative volumes of traffic involved in various turning and through movements.

10.7.1.1 Auxiliary Right-Turn Lanes

The use of auxiliary right-turn lanes at intersections can significantly improve operations. However, in urban areas the designer needs to consider the effect the wider pavement has on pedestrian crossings. Right-turn lanes should be considered after a traffic capacity analysis indicates their need.

For rural areas, consider using an exclusive right-turn lane:

- at any unsignalised intersection on a two-way single carriageway that satisfies the criteria in Figure 10-36;
- at any unsignalised intersection on a high-speed, dual carriageway that satisfies the criteria in Figure 10-37;

- on expressways at all public road intersections where the current ADT on the side road is greater than 250;
- at any intersection where a capacity analysis determines a right-turn lane is necessary to meet the level-of-service criteria;
- at any signalised intersections where the right-turning volume is greater than 150 vph and where there is greater than 300 vphpl on the mainline;
- for uniformity of intersection design along the roadway if other intersections have right-turn lanes;
- at any intersection where the mainline is curved to the left and where the mainline curve requires superelevation;
- at railroad crossings where the railroad is located close to the intersection and a right-turn lane would be desirable to efficiently move through traffic on the parallel roadway; or
- at any intersection where the crash experience, existing traffic operations, sight distance restrictions (e.g. intersection beyond a crest vertical curve), or engineering judgment indicates a significant conflict related to right-turning vehicles.

For urban areas, in addition to traffic analysis, the above criteria can also be selectively applied for considering using an exclusive right-turn lane.

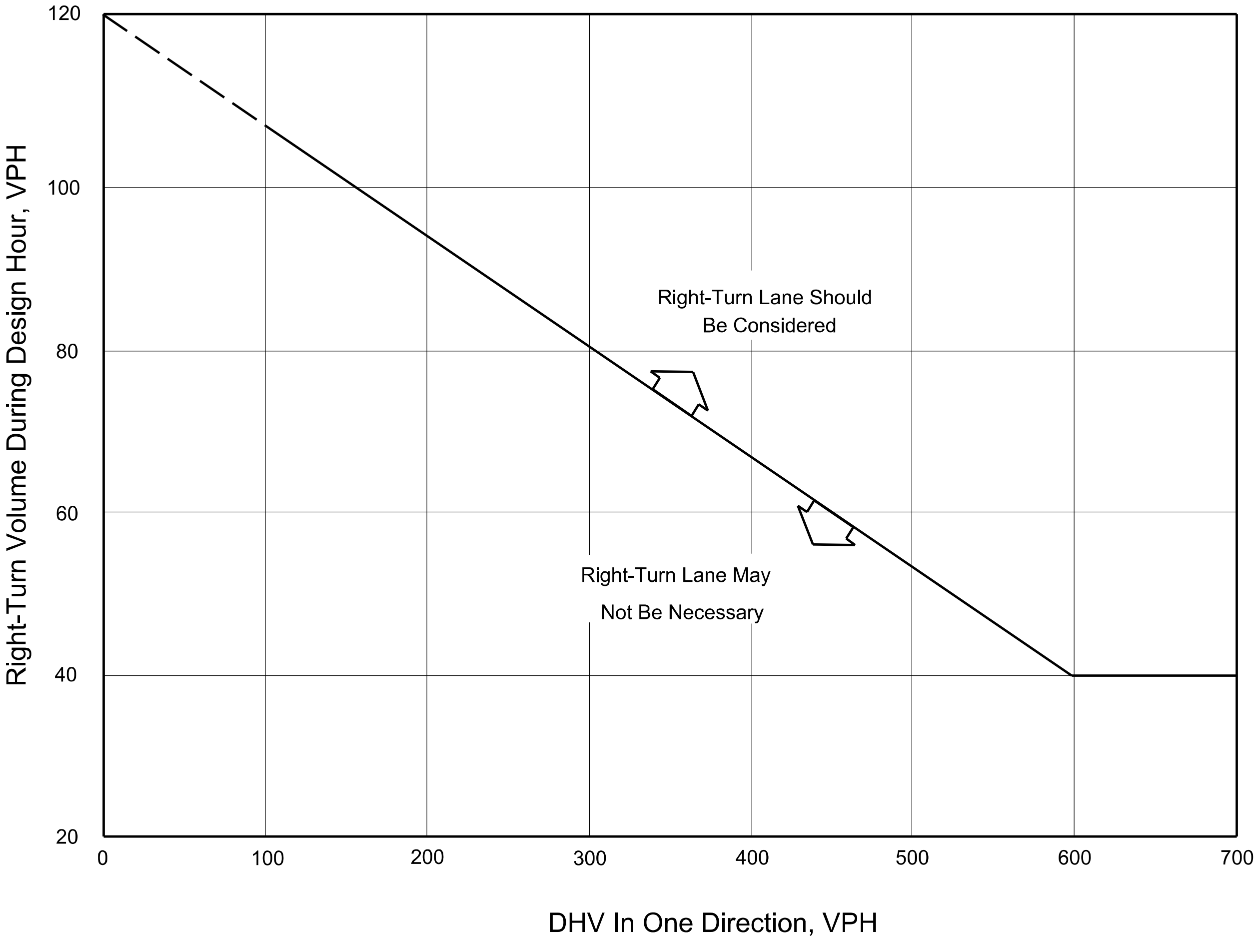
10.7.1.2 Auxiliary Left-Turn Lanes

In designing an intersection, left-turning traffic should be removed from the through lanes, whenever practical. Therefore, provisions for left turns (i.e. auxiliary left-turn lanes) have widespread application. Ideally, left-turn lanes should be provided at driveways and street intersections along major arterial and collector roads wherever left turns are permitted. In some cases or at certain locations, providing for indirect left turns (i.e. jug-handles, U-turn lanes, and diagonal roadways) may be appropriate to reduce crash frequencies and preserve capacity. Left-turn facilities should be established on roadways where traffic volumes are high enough or crash histories are sufficient to warrant them. They are often needed to provide adequate service levels for the intersections and the various turning movements.

The *Highway Capacity Manual 2010* (17) recommends that exclusive left-turn lanes at signalised intersections should be installed as follows:

- Exclusive left-turn lanes should be provided where exclusive left-turn signal phasing is provided.
- Exclusive left-turn lanes should be considered where left-turn volumes exceed 100 veh/h (left-turn lanes may be provided for lower volumes based on assessment of the need).
- Double left-turn lanes should be considered where left-turn volumes exceed 300 veh/h.

Figure 10-36: Guidelines for Right-Turn Lanes at Unsignalised Intersections on Two-Way Single Carriageway



Note: *For roadways with a design speed below 80 km/h, with a DHV in one direction of less than 300, and where right turns are greater than 40, an adjustment should be used. To read the vertical axis of the chart, subtract 20 from the actual number of right turns.*

Source: (48)

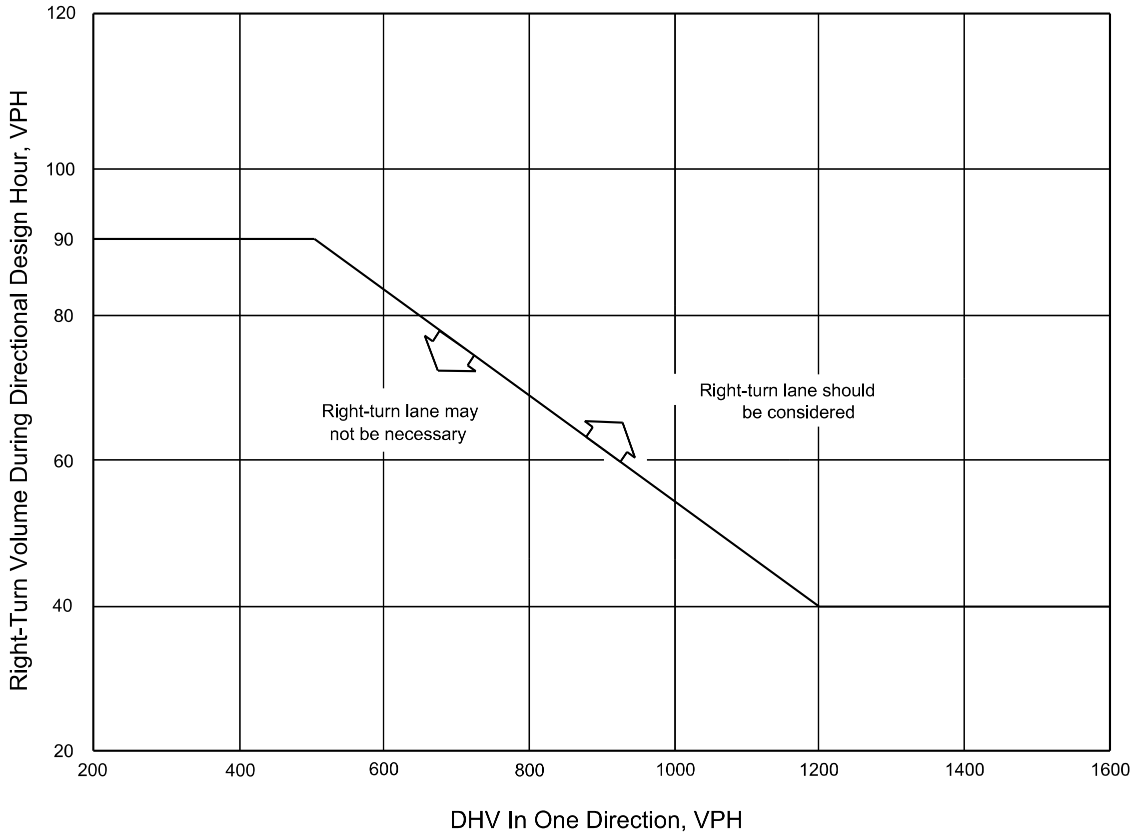
Example 10.7-1

Given:	Design Speed	=	60 km/h
	DHV (in one direction)	=	250 vph
	Right Turns	=	100 vph

Problem: Determine if a right-turn lane is warranted.

Solution: To read the vertical axis, use 100 – 20 = 80 vph. Figure 10-36 indicates that right-turn lane is not necessary, unless other factors (e.g. high crash rate) indicate a lane is needed.

Figure 10-37: Guidelines for Right-Turn Lanes at Unsignalised Intersection on Dual Carriageway (80 km/h or greater)



Note: For speeds less than 80 km/h, see Section 10.7.1.1.

Source: (48)

Example 10.7-2

Given:	Design Speed	=	90 km/h
	DHV (in one direction)	=	800 vph
	Right Turns	=	80 vph

Problem: Determine if a right-turn lane is warranted.

Solution: Figure 10-37 indicates that for the turning and through volumes given, a right-turn lane should be considered.

Table 10-13 provides traffic volumes where left-turn facilities should be considered on two-lane roadways. The left- and right-turn volumes from the minor road can be equal to, but not greater than, the left-turn volumes from the major road shown in the Table 10-13.

Table 10-13: Guide for Left-Turn Lanes on Two-Way Single Carriageway

Opposing Volume (veh/h)	Advancing Volume (veh/h)			
	5% Left Turns	10% Left Turns	20% Left Turns	30% Left Turns
60-km/h Operating Speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
80-km/h Operating Speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295
100-km/h Operating Speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Source: (1)

Example 10.7-3

Given:
Design Speed = 80 km/h
Advancing Volume = 400 vph
Opposing Volume = 400 vph
Left Turns = 10%

Problem: Determine if a left-turn lane is warranted.

Solution: Table 10-13 indicates that for an opposing volume of 400 veh/h, the advancing volume should 320 veh/h or less; therefore, a left-turn lane should be considered.

10.7.2 Design of Turn Lanes

10.7.2.1 Turn Lane Widths

The width of the turn lane should be determined relative to the functional class, urban/rural location, and project scope of work.

Desirably, rural auxiliary turn-lane widths should be 3.65 m, with a minimum width of 3.3 m. In general, the minimum shoulder widths adjacent to a turn lane with shoulders should be 1.2 m.

In urban areas, desirable auxiliary turn-lane widths may be 3.3 m with a minimum of 3.0 m, provided the turning truck volume is low.

10.7.2.2 Turn Lane Lengths

Desirably, the length of a right- or left-turn lane at an intersection should allow for both safe vehicular deceleration and storage of turning vehicles outside of the through lanes. However, this is often not practical. The length of auxiliary lanes will be determined by a combination of its taper length (L_T), deceleration length (L_D), and storage length (L_S). For urban areas, the functional length will be the taper length plus the storage length or the deceleration length plus the taper length, whichever is larger. For rural areas, typically the functional length will be the deceleration length that includes the taper length. In most high-speed, low-volume rural situations, the storage length will not be a controlling factor. Figure 10-38 illustrates a schematic of auxiliary lanes at an intersection. This can also be adapted for right-in/right-out (entry/exit) junctions in urban areas.

The following discusses the criteria for turn-lane lengths:

1. Taper. The entrance taper into the turn lane may be either a straight or a reverse curve taper. Where the roadway is on a curved alignment, the taper of the turn lane should be more pronounced than usual to ensure that the through motorists are not inadvertently directed into the turn lane. This is accomplished by shortening the taper length. Where the entrance taper is shortened, ensure that the overall deceleration distance is still provided for the turn lane. Taper designs may be one of the following:

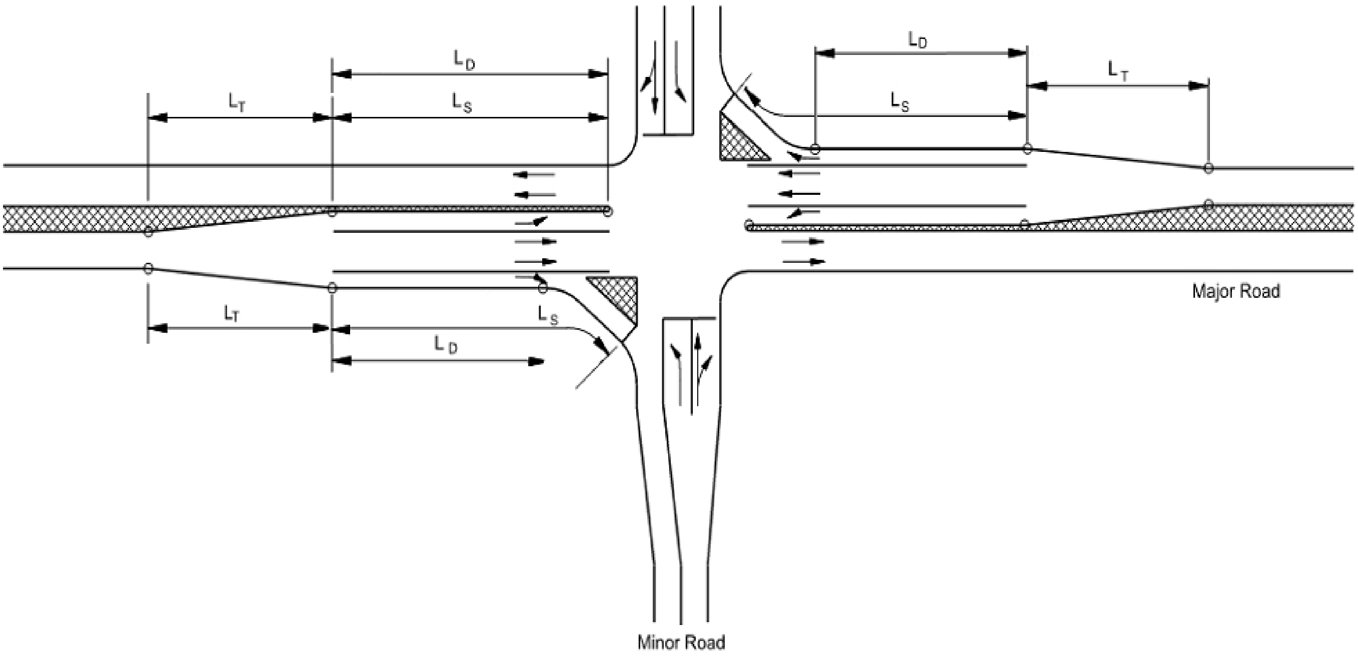
   a. Straight-Line Tapers. Straight –line tapers (Figure10-38 and Figure 10-39(a)) are the easiest to design and construct. For rural roads, the taper rate may be 8:1 for design speeds up to 50 km/h and 15:1 for design speeds of 80 km/h and greater. For urban roads, taper length for single turn lane is 30m and for dual turn lane is 45m. Straight-line tapers are particularly applicable where a paved shoulder is striped to delineate the auxiliary lane. Always use straight tapers across bridges for ease of construction. Do not use short, straight-line curbed tapers because of the probability of vehicles hitting the leading end of the taper.

   b. Partial Straight-Line Tapers. A short curve is desirable at each end of long tapers as shown in Figure 10-39(b), but may be omitted for ease of construction. Where curves are used at the ends, the tangent section should be about one-third to one-half of the total length.

   c. Symmetrical Reverse Curve Tapers. This taper design is most commonly used on curbed urban streets. Figure 10-39(c) shows a design taper with symmetrical reverse curves.

   d. Asymmetrical Reverse Curve Tapers. This taper design is the most desirable; see Figure 10-39(d). For this design, the turnoff curve radius is about twice that of the second curve. Where 30 m or more in length is provided for the tapers, see Figure 10-39(d), use options 1 or 2 for low-speed operations.

Figure 10-38: Typical Auxiliary Lanes at Intersections



Note: The schematic of the major road (free flowing) also applies to all legs of a signalised intersection.

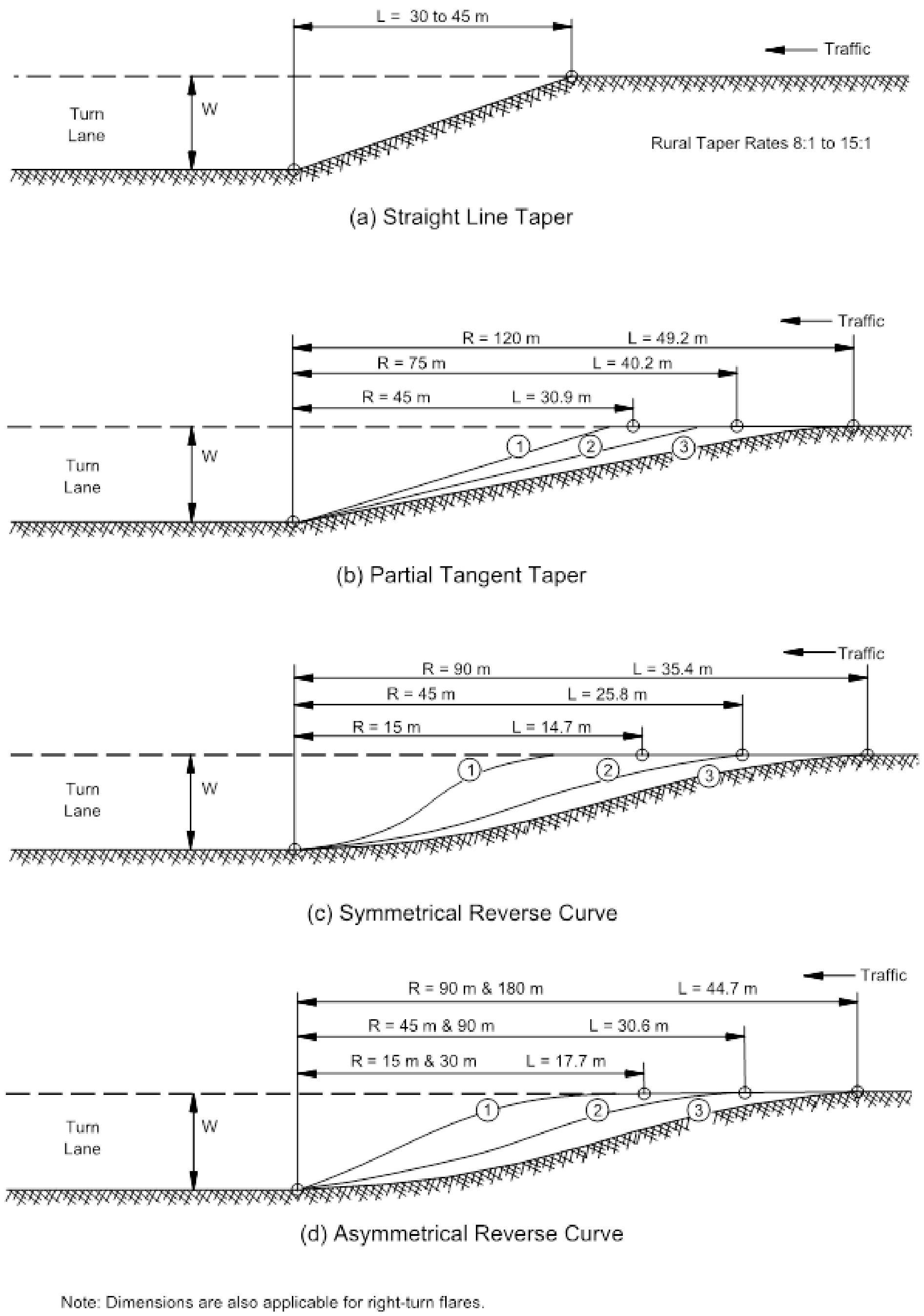
Key: L_T = *Taper length*

L_D = *Deceleration length*

L_S = *Storage length*

See Section 10.7.2 for additional guidance.

Figure 10-39: Examples of Taper Designs



Source: (1)

All the example design dimensions and configurations shown in Figure 10-39 are applicable to right-turn lanes as well as left-turn lanes.

2. Deceleration. For rural facilities, the deceleration distance (L_D) should meet the criteria presented in Section 12.6.1.2 for ramp exit terminals. The following will apply:

   a. Design Speed. The deceleration length will depend upon the mainline design speed and the proposed type of operation at the end of the turn lane.

b. Location. The deceleration distance includes the taper lengths. For left turns, the deceleration distance is usually measured beginning at the end of the left-turn control radii. For right turns, the deceleration distance may be set at either one of two locations – at the beginning of the curve radius or at the stop bar for turning roadways. See Figure 10-38. At intersections with minor public roads (e.g. frontage roads, service drives, local roads with current ADT volumes less than 1500), a design speed of 80 km/h may be used to determine the deceleration length.

c. Urban. Distances shown in Section 12.6.1 may be applied on urban facilities; however, this is not always feasible due to restricted urban conditions. Length of deceleration lanes for urban intersections may be suitably adjusted as per the design speed and/or traffic studies justifying additional storage lane requirement. Taper length to be 30 m instead of 75 m. The below values are recommended where deceleration/storage lanes are required at Signalized Intersections and where right-in right-out located on urban expressways and arterial roads.

Design Speed (kph)	Painted Nose for RIRO's Length (m)	Deceleration Lane Length (m) (Excluding Taper & Nose)	Taper Length (m)
60	-	25 to 40	30
80	-	55 to 80	30
100	20	80 to 110	30
120	40	100 to 120	30

Recommended Deceleration Lane Lengths in Urban areas

d. Trucks. Where it is determined that a turn lane will be used by a large number of trucks, increase the length of the deceleration distance by approximately 30%. This will compensate for the braking constraints of large trucks.

3. Storage Length (Signalised Intersections). The storage length (L_S) for turn lanes should be sufficient to store the number of vehicles likely to accumulate during the red phase of the signal cycle in the design hour. The designer should consider the following in determining the storage lengths for signalised intersections:

a. Determine the distance using the criteria for signalised intersections in the *Highway Capacity Manual 2010* (17).

b. Where right-turns-on-red are permitted or where separate right-turn signal phases are provided, the length of the right-turn lane may be reduced due to less accumulation of turning vehicles. The storage length (LS) needed for a separate right-turn lane is measured from the stop bar for the right-turning roadway; see Figure 10-38.

c. At signalised intersections, the designer should also consider that entry into right- and left-turn lanes might be blocked for the signal storage queue of the adjacent through lanes. If this occurs, provide longer lengths of turn lanes.

d. The minimum storage length for urban auxiliary lanes is 24 m.

4. Storage Length (Unsignalised Intersections). To determine the minimum storage length for unsignalised intersections, assume that the intersection is signalised with a two-phase signal using a 40-second to 60-second cycle length. Then use the *Highway Capacity Manual 2010* (17) to determine the expected storage length.

10.7.3 Design of Left-Turn Lane

10.7.3.1 General Criteria

In addition to the criteria for left-turn lane widths and lengths discussed in Section 10.7.2, the designer should consider the following general criteria:

1. Transition Areas. Do not locate left-turn lanes within any portion of a channelized approach island that is transitional in width.

2. Taper Design. Figure 10-38 and Figure 10-40 illustrate the use of a straight-line taper. Figure 10-41 illustrates the use of reverse curves for an entrance taper.

3. Offset Turn Lanes. Providing an offset design ensures that opposing left-turning motorists can see past one another to view oncoming through traffic. Offset left-turn lanes can be either a parallel or a tapered type; see Section 10.7.3.3.

4. Indirect Turns. Where operational or safety concerns preclude the use of typical left-turn lanes, the designer may consider the use of indirect left turns or jug handles that cross the mainline or intersect the crossroad at a different location

5. Opposing Left-Turning Traffic. If simultaneous and opposing left-turn lanes are proposed, the designer must ensure that there is sufficient space for all turning movements. Desirably, the separation between pavement markings should be 3 m. If space is unavailable, it will be necessary to alter the signal phasing to allow the two directions of turning traffic to move through the intersection on separate phases. See Section 10.7.3.4 for additional guidance.

6. Phasing. Only use lagging left turns (i.e. after the through phase of a signal). For additional guidance, see the Abu Dhabi Manual *on Uniform Traffic Control Devices* (22).

7. Pedestrian Refuges. On Boulevards and Avenues, ensure there is a minimum 2.0 m pedestrian refuge in the median when introducing left turn lanes; see Figure 10-33.

10.7.3.2 Parallel Left-Turn Lanes without Offset

Figure 10-38, **Figure 10-40**, and Figure 10-41 and the following provide the design criteria for left-turn lanes that are adjacent and parallel to the through travelled way and are not offset:

1. Two-Lane Facilities. A channelized left-turn lane with a flush island or raised island may be used depending on specific site conditions; see **Figure 10-40**.

2. Narrow Raised Medians. Left-turn lanes generally will be the parallel design. This design is illustrated in **Figure 10-40** and Figure 10-41.

Figure 10-40: Flush Channelized Islands at Isolated High-Speed Rural Intersection

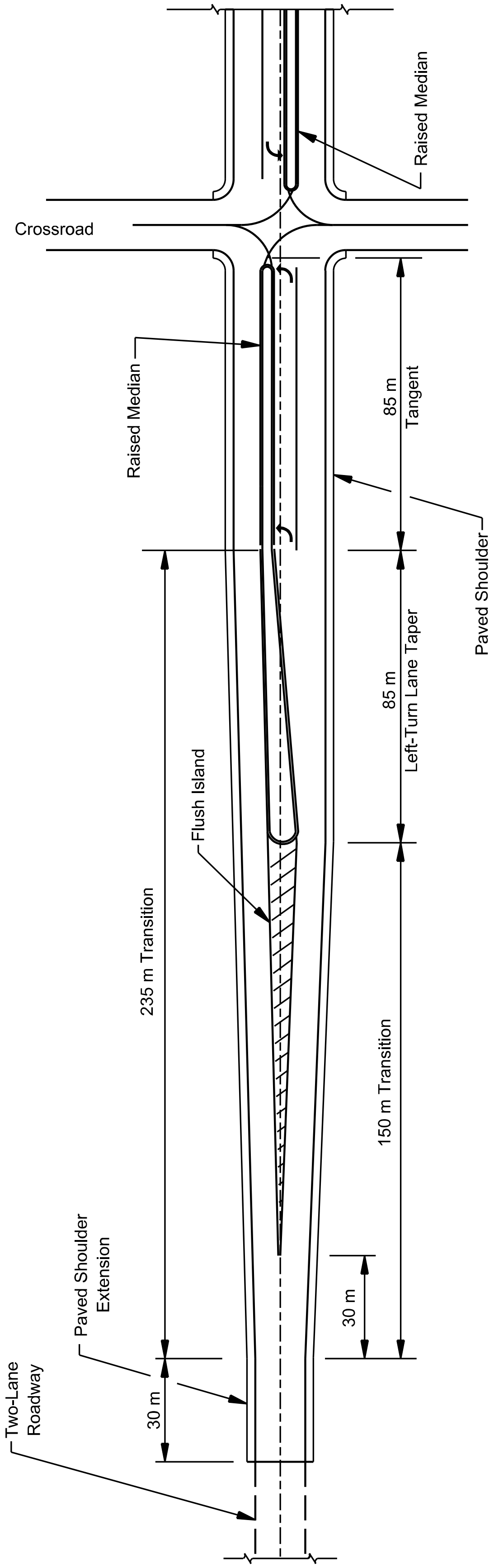
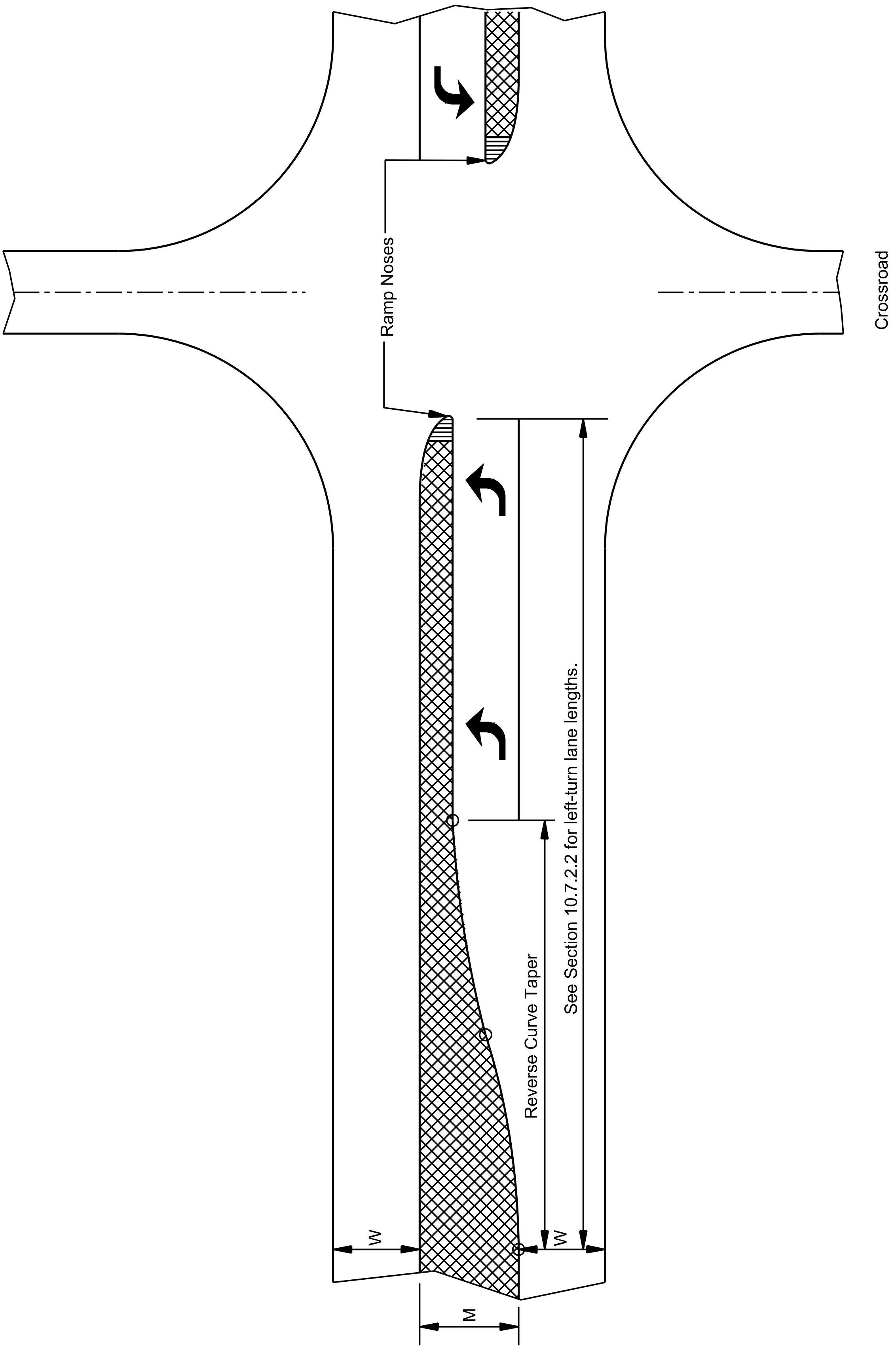


Figure 10-41: Raised Channelized Intersection (Parallel Left-Turn Lane)



3. Narrow Expressway Medians. Left-turn lanes generally will be the parallel design due to restricted right of way.

4. Multilane Roadways with Wide Medians. The designer should use a parallel left-turn lane design with wide medians.

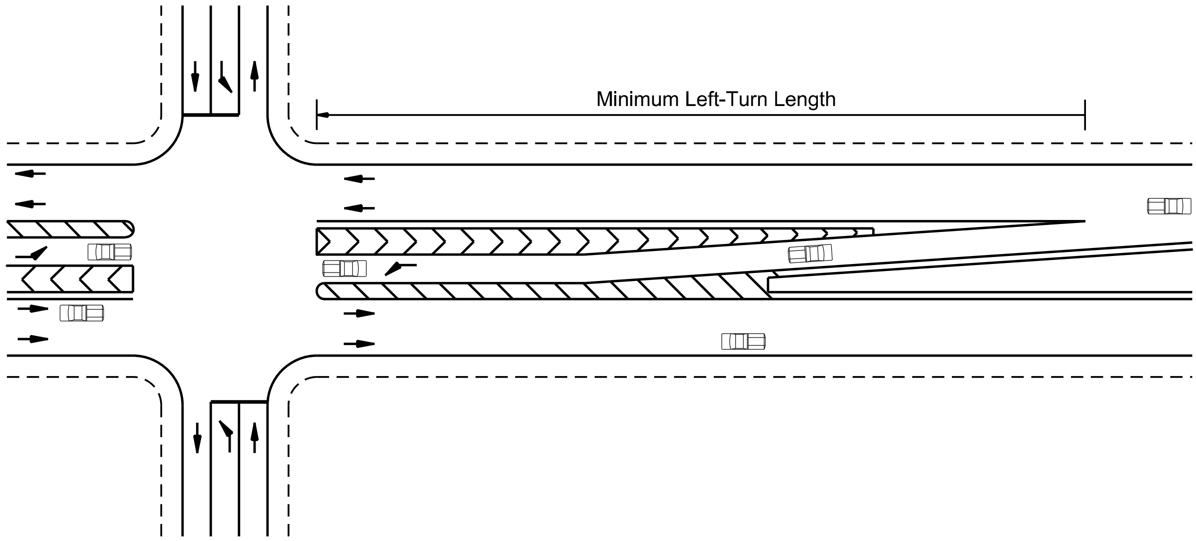
10.7.3.3 Offset Left-Turn Lanes

For medians wider than about 5.5 m, it is desirable to offset the left-turn lane so that the width of the divider between opposing lane is reduce to 1.8 m to 2.4 m rather than to align it exactly parallel with and adjacent to the through lane; see Figure 10-42. This alignment places the vehicle waiting to make the left turn as far to the left as practical, maximising the offset between the opposing left-turn lanes, and thus providing improved visibility of opposing through traffic. The advantages of offsetting the left-turn lanes are (1) better visibility of opposing through traffic; (2) decreased possibility of conflict between opposing left-turn movements within the intersection; and (3) more left-turn vehicles served in a given period of time, particularly at a signalised intersection. Parallel offset left-turn lanes may be used at both signalised and unsignalised intersections. This left-turn lane configuration is referred to as a parallel offset left-turn lane and is illustrated in Figure 10-43(a).

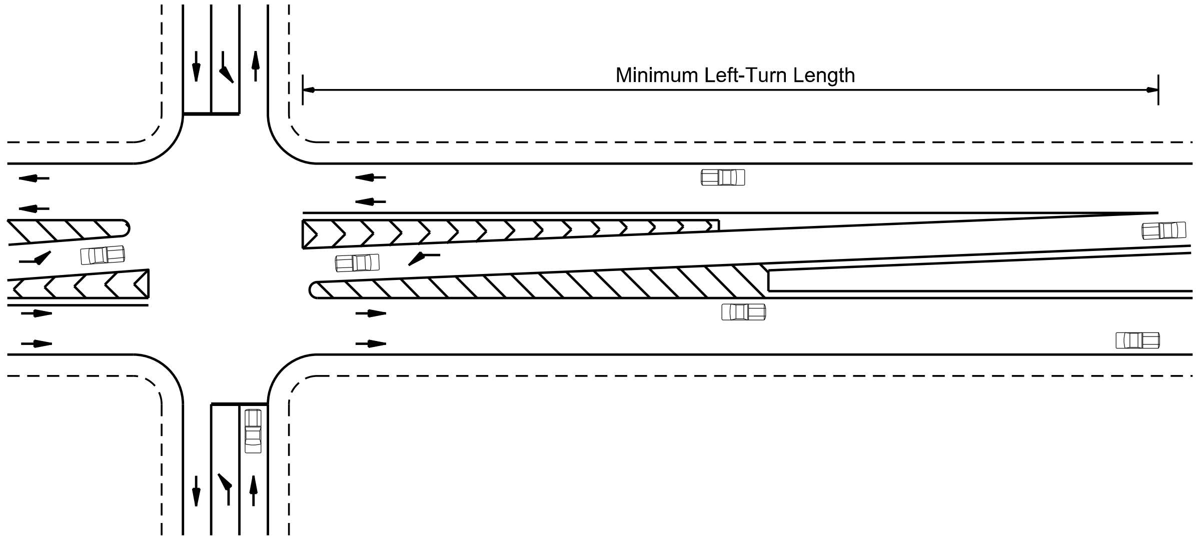
An offset between opposing left-turn vehicles can also be achieved with a left-turn lane that diverges from the through lanes and crosses the median at a slight angle. Figure 10-43(b) illustrates a tapered offset left-turn lane of this type. Tapered offset left-turn lanes provide the same advantages as parallel offset left-turn lanes in reducing sight distance obstructions and potential conflicts between opposing left-turn vehicles and in increasing the efficiency of signal operations. Tapered offset left-turn lanes are normally constructed with a 1.2-m nose between the left-turn lane and the opposing through lanes. Tapered offset left-turn lanes have been used primarily at signalised intersections.

Figure 10-42: Intersection with Parallel Offset Left-Turn Lanes

Figure 10-43: Parallel and Tapered Offset Left-Turn Lane



(a) Parallel-Offset Turn Lane



(b) Tapered-Offset Turn Lane

This type offset is especially effective for turning radii allowance where trucks with long rear overhangs are turning from the mainline roadway. This same type of offset geometry may also be used for trucks turning right with long rear overhangs.

Figure 10-44, taper design, and Figure 10-45, parallel design, illustrate example designs of offset left-turn lanes. Each intersection should designed on case-by-case basis considering traffic volumes, turning volumes, median type and width, and available right-of-way. In addition, the designer should consider the following:

1. Tapered Offset Left-Turn Lanes. Figure 10-44 illustrates a typical tapered offset left-turn lane design within a rural wide median. The advantages of the tapered offset design versus a parallel lane design without an offset is that the offset design provides better visibility for the turning motorist to the opposing traffic, decreases the possible conflict between opposing left-turning vehicles, and serves more left-turning vehicles in a given time period. In addition, the designer should consider the following:

   a. Guidelines. Provide a tapered offset left-turn lane design where at least two of the following are applicable:

      - the median width is equal to or greater than 12 m and only one left-turn lane in each direction on the mainline roadway is required for capacity;
      - the current mainline ADT is 1500 or greater and the left-turn DHV in each direction from the mainline is greater than 60 vph. Under these conditions, vehicles waiting in opposing left-turn lanes have the probability of obstructing each other's line of sight; and
      - the intersection will be signalised.

   b. Median Widths. Desirably, the median width should be 15 m.

   c. Curb and Gutter. Use curb and gutter on all corner and channelizing island.

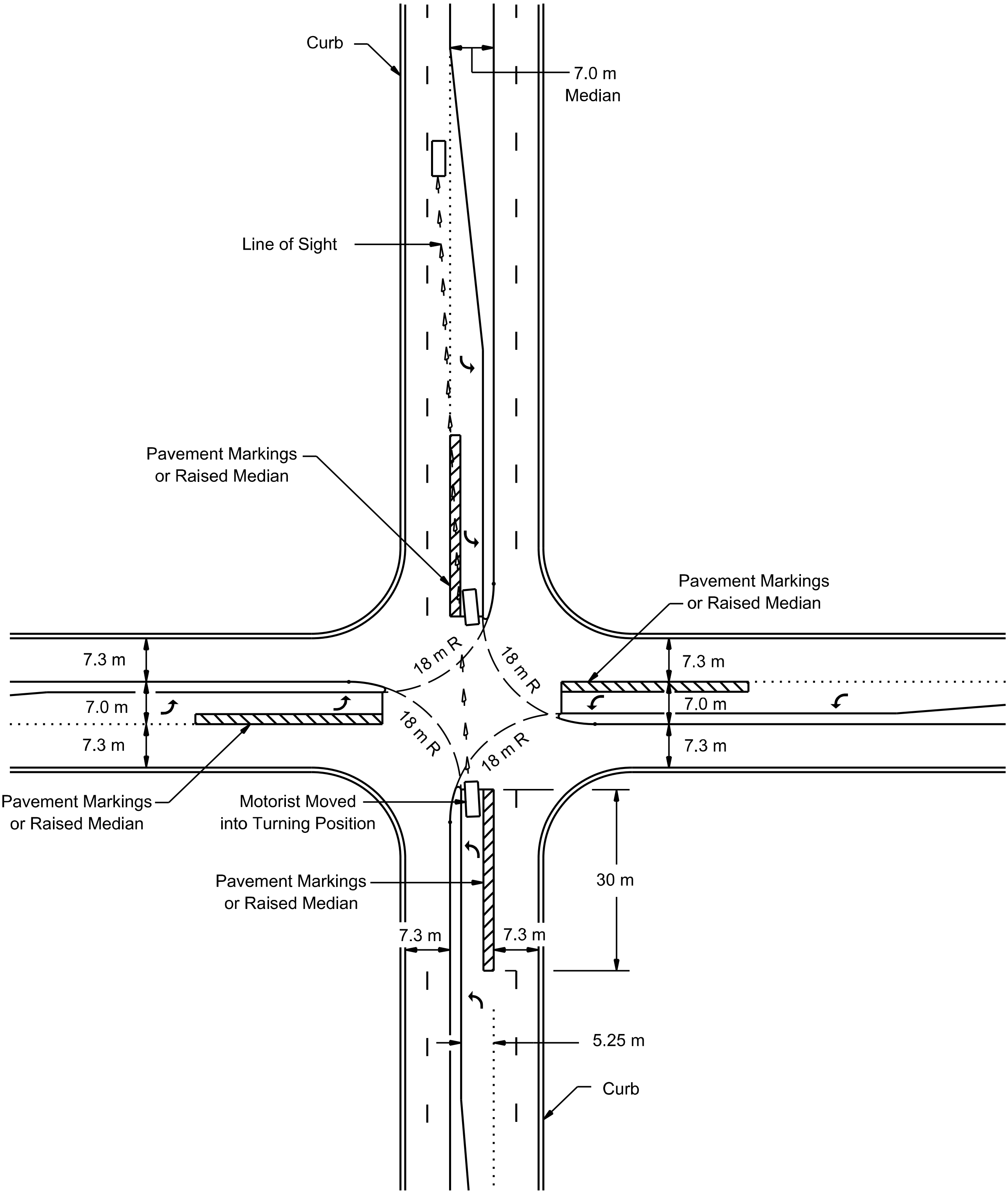
2. Parallel Offset Left-Turn Lanes. Parallel offset left-turn lanes offer the same advantages as the tapered design. However, they may be used at intersections with medians less than 12 m but greater than 4.0 m. Figure 10-42 illustrates intersections with parallel offset left-turn lanes.

10.7.3.4 Simultaneous Left Turns

Simultaneous left turns may be considered at an intersection of two major roadways; however, simultaneous opposing trucks on single-lane left-turn lanes is generally impractical. Figure 10-46 indicates traffic patterns that should be considered in the design. Pavement marking details are given in the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

A design feature that can improve intersection operation is to provide a minimum clear distance of 3.0 m between the two opposing left-turn movements within the intersection.

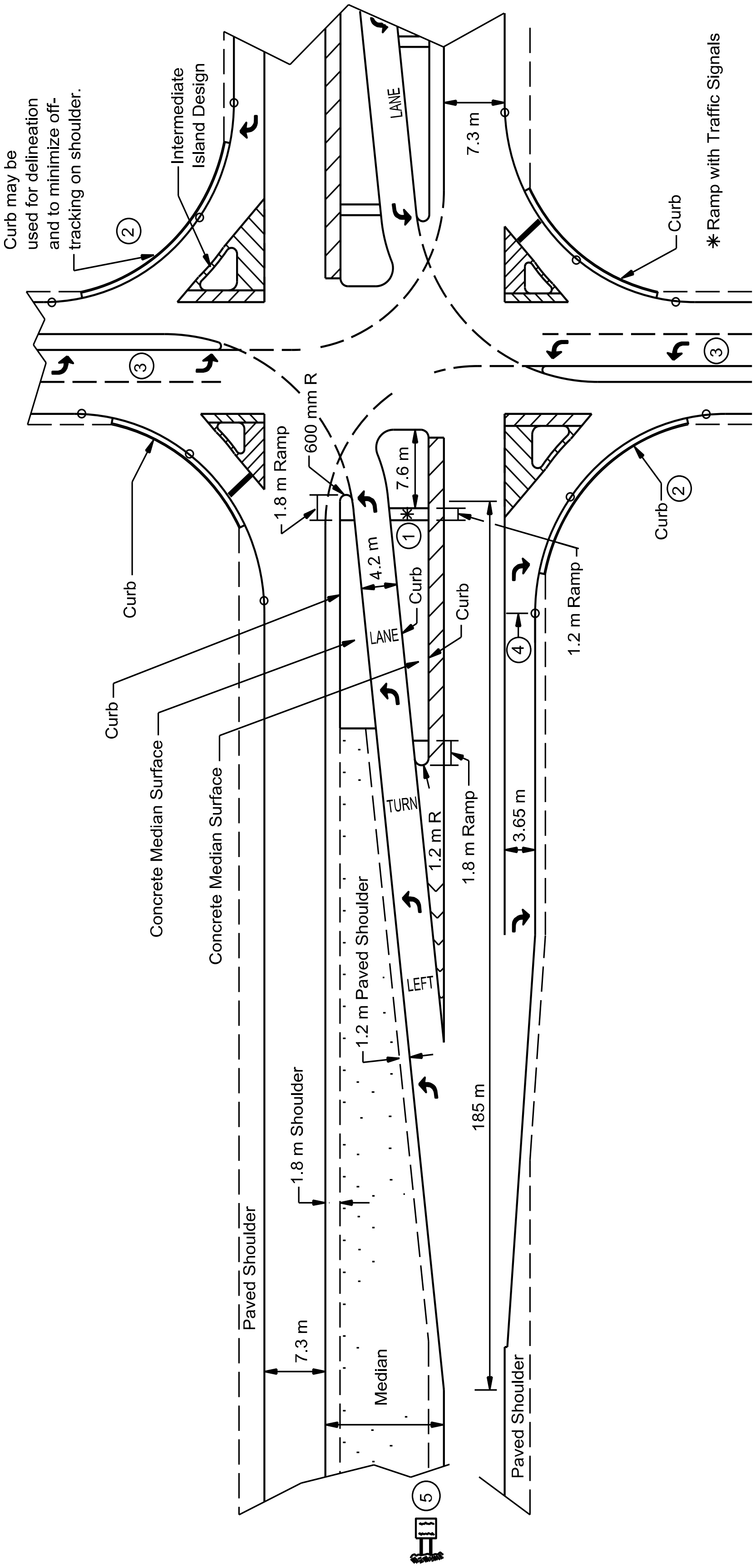
Figure 10-44 Typical Design for Parallel Offset Left-Turn Lanes



Note: For urban streets refer to corresponding lane widths

Figure 10-45: Example of a Tapered Offset Left-Turn Lane Design

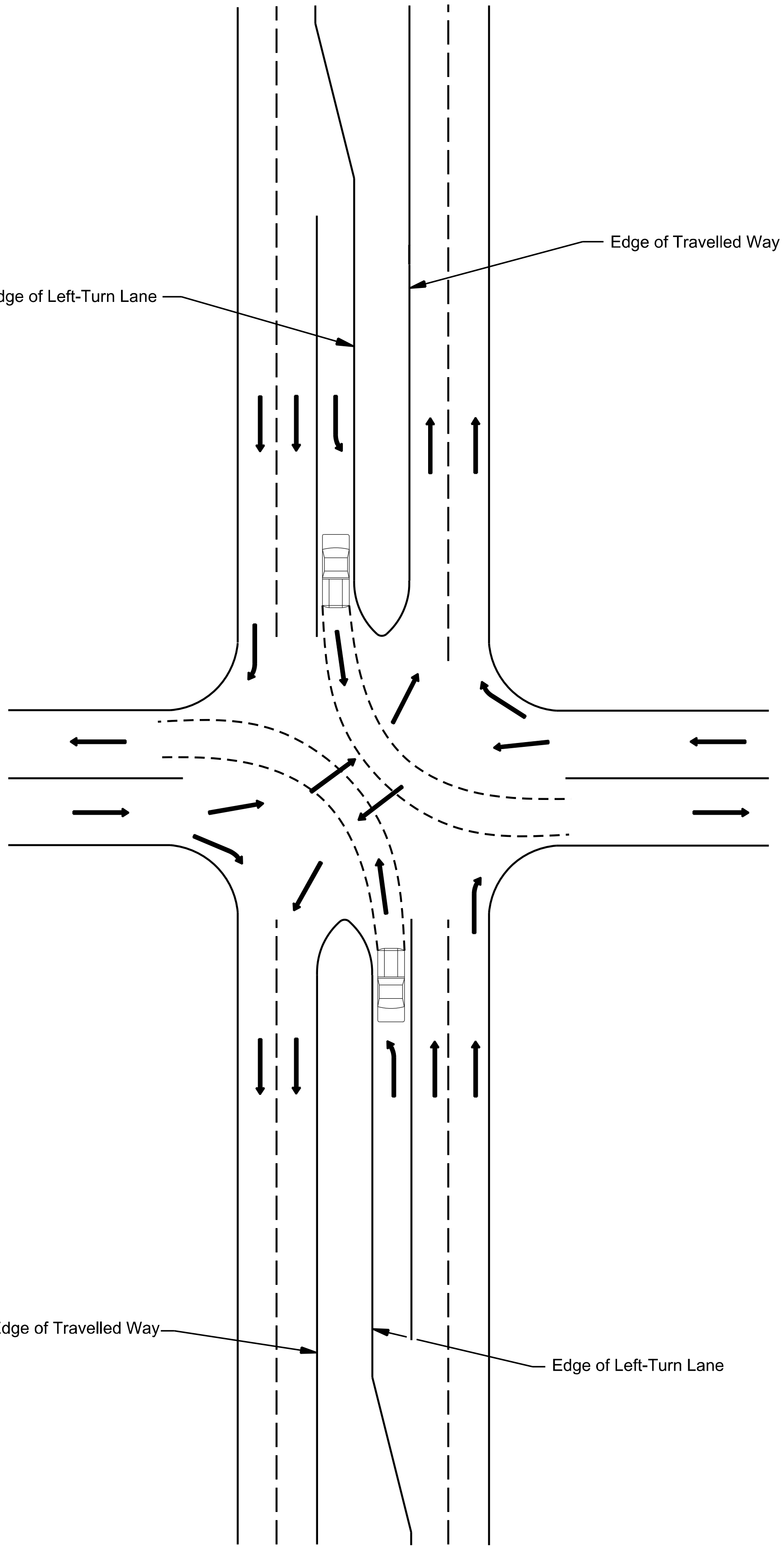
Note: For urban streets refer to corresponding lane widths



Notes:

1. *Place drainage inlets in the gutter to facilitate drainage within the left-turn bay.*
2. *A three-centred curve is desirable with right-turn lanes.*
3. *Where a left-turn lane is required, design the median as a raised median for delineation. Where no median is proposed, the minimum travelled way width should be 7.3 m.*
4. *End the right-turn deceleration length at beginning of the radius turn.*
5. *See Abu Dhabi Manual on Uniform Traffic Control Devices (22) for proper placement of guide sign.*

Figure 10-46: Four-Leg Intersection Providing Simultaneous Left Turns



10.7.4 Design of Right-Turn Lane

Section 10.7.2 provides design criteria for right-turn lane widths and lengths. Right-turn lanes may be designed with or without turning roadways depending on site conditions. Figure 10-38 and Figure 10-45 illustrate typical designs for right-turn lanes. Figure 10-47 shows an intersection with right-turn lanes. Because of potential conflicts with right-turning traffic, entrances should not be allowed within the limits of the right-turn lane as shown in the lower, left-hand corner of Figure 10-47.

In urban areas, minimise the use of dedicated right-turn lanes and right-turn slip lanes. Section 10.4.2.2 identifies the prefer options for right turns, in priority order. A traffic analysis (software simulation) of both the intersection turning movements and the localised network is required for all but Option 1 in Section 10.4.2.2.

Figure 10-47: Intersection with Right-Turn Lanes

10.7.5 Multiple Turn Lanes

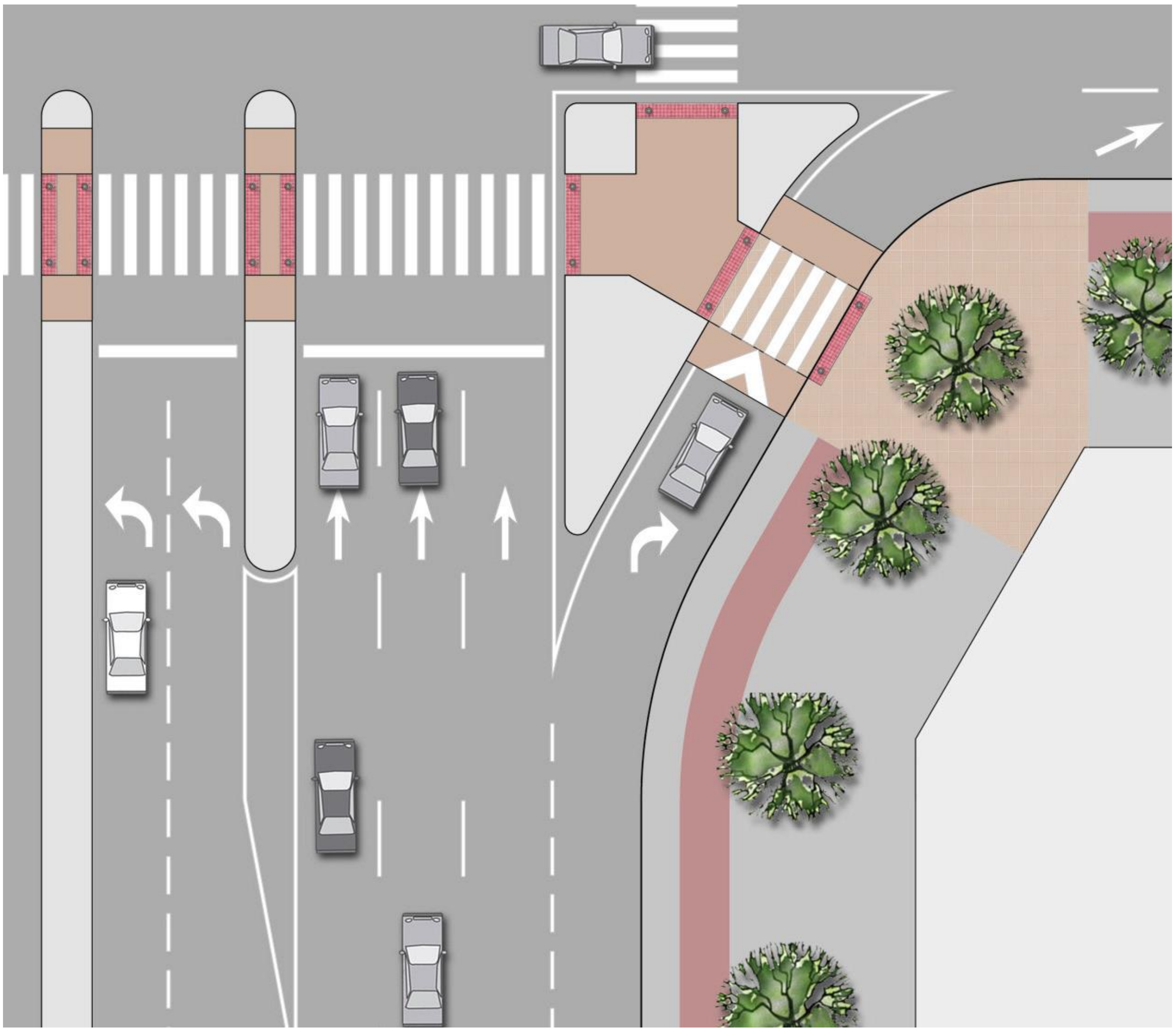
10.7.5.1 Guidelines

At intersections with high-turning volumes, multiple left- and/or right-turn lanes may be considered to reduce delay, increase capacity, and increase the level of service at an intersection. However, multiple turn lanes may cause problems with right-of-way, lane alignment, for crossing pedestrians, and erratic movements for turning drivers. As additional lanes are added, pedestrians require more crossing time and additional safety refuge medians; see Figure 10-48.

Multiple left- and/or right-turn lanes are generally only considered where:

- there is insufficient space to provide the necessary length of a single-turn lane because of restrictive site conditions (e.g. closely spaced intersections);

Figure 10-48: Pedestrian Refuge at Multiple Left-Turn Lanes



Source: (4)

- based on a capacity analysis, the necessary time for a protected left-turn phase for a single lane becomes unattainable to meet the level-of-service criteria (average delay per vehicle); and/or
- more than 300 vph are projected to be turning.
- For operational purposes, only provide urban dual-left turn lanes to reduce delay at a signal, but not to increase capacity. At some intersections, triple left-turn lanes may be required. These should only be used after ensuring alternative solutions are not feasible.
- In may be necessary to use dual right-turn lanes. However, their use is not recommended. A traffic analysis should clearly demonstrate the benefits and necessity of the additional lanes.
- In extreme conditions, triple left-turn lanes may be necessary for capacity; however, they should only be considered after considering that alternative solutions are not feasible and with the approval of the Agencies. In such cases, give special consideration to providing pedestrian crossings and median refuges.

Multiple turn lanes should only be used with signalisation providing a separate turning phase. The use of multiply turn lanes in urban areas will require approval by the client.

10.7.5.2 Double and Triple Left-Turn Lanes

Where two median lanes are provided as double left-turn lanes, left-turning vehicles leave the through lanes to enter the median lanes in single file, but once within the median lanes, the vehicles are stored in two lanes. On receiving the green indication, the left-turning vehicles turn simultaneously from both lanes. With three-phase signal control, such an arrangement results in an increase in capacity of approximately 180% of that of a single median lane.

Occasionally, the two-abreast turning manoeuvres may lead to sideswipe crashes. These usually result from too sharp a turning radius or a roadway that is too narrow.

Figure 10-49 illustrates the more important design elements for dual left-turn lanes. Figure 10-50 shows an intersection with dual left-turn lanes. In addition, the designer should consider the following:

1. Turning Radii. Off tracking and swept path width are important factors in designing double and triple left-turn lanes. Vehicles should be able to turn side-by-side without encroaching upon the adjacent turn lane. A desirable turning radius for double or triple left-turn lanes is 25 m, which will accommodate the P, SU-9, or SU-12 design vehicles within a swept path width of 3.6 m. Larger vehicles need greater lane widths to negotiate double or triple left-turn lanes constructed with a 25 m turning radius without encroaching on the paths of vehicles in the adjacent lane. Table 10-14 illustrates the swept path widths for specific design vehicles making 90-degree left turns. Table 10-14 can be used to determine the width needed at the centre of a turn where the maximum vehicle off tracking typically occurs. To help drivers maintain their vehicles within the proper lanes, the longitudinal lane line markings of double or triple-left lanes may be extended through the intersection area to provide positive guidance. This type of pavement marking extension is intended to provide a visual cue for lateral positioning of the vehicle as the driver makes a turning manoeuvre.

2. Throat Width. The receiving leg of the intersection should have adequate width to accommodate two lanes of turning traffic. A minimum width of 9 m is used by several roadway agencies.

3. Median Widths. Dual left-turn lanes will require a median width of at least 9.0 m.

4. Special Pavement Markings. As illustrated in Figure 10-49 pavement markings can effectively guide two lines of vehicles turning abreast. See the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) for applicable guidelines on the selection and placement of any special pavement markings.

Figure 10-49: Schematic for Dual Left-Turn Lanes

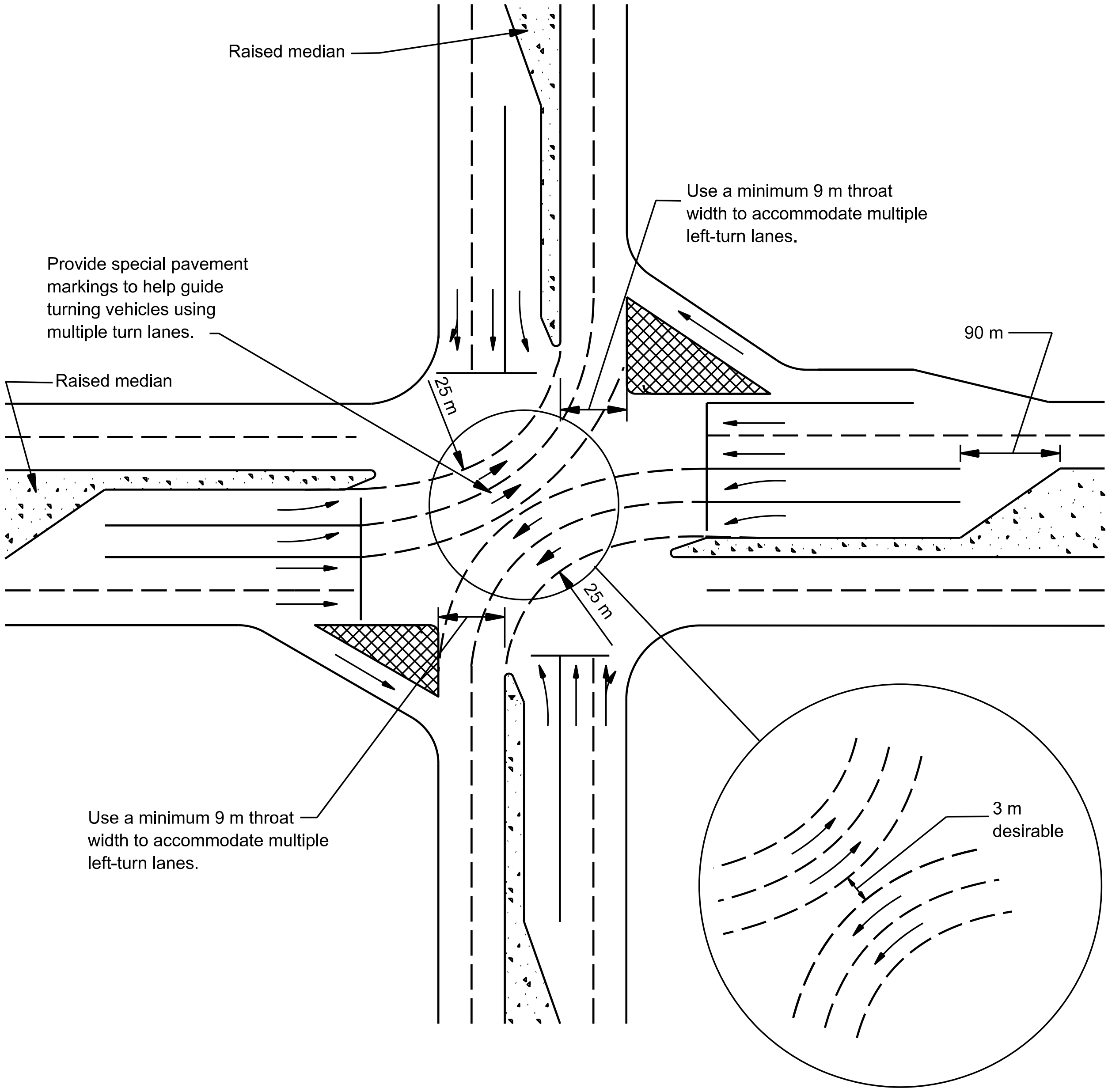


Figure 10-50: Intersection with Dual Left-Turn Lanes

Table 10-14: Swept Path Width for 90-Degree Left Turns

Turning Radius from Turning Lane (m)	Swept Path Width (m) for Specific Design Vehicles		
	SU-9	SU-12	WB-20
23	3.3	3.7	6.4
30	3.0	3.4	5.6
46	2.8	3.1	4.6

Source: (1)

5. Opposing Left-Turning Traffic. If simultaneous and opposing dual left-turn lanes are proposed, the designer must ensure that there is sufficient space for all turning movements. Desirably, the separation between pavement markings should be 3 m; see Figure 10-49. If space is unavailable, it will be necessary to alter the signal phasing to allow the two directions of turning traffic to move through the intersection on separate phases.

6. Turning Templates. All intersection design elements for multiple turn lanes must be checked by using a computer simulated turning template program. The designer should assume that the selected design vehicle will turn from the outside lane of the multiple turn lanes. Desirably, the inside vehicle should be an SU-9 but, as a minimum, the inside vehicle can be assumed to be a passenger car turning side by side with the selected design vehicle.

10.8 Median Openings

10.8.1 Location/Spacing

Desirably, median openings should be provided on divided roadways at all public roads and major traffic generators. However, this may result in close intersection spacing that may impair the operation of the facility. The minimum spacing is provided in the Abu Dhabi *Access Management Policy and Procedures Manual* (21).

For both rural and urban facilities, the available sight distance near a median opening is also a factor in the determination of its location. In addition, on some facilities, commercial establishments with heavy truck traffic may dictate the location of median openings.

10.8.2 Design

Figure 10-51 presents a general figure for the design of a median opening at an intersection. The following will apply to the design of median openings:

1. Design Vehicle. Use the largest vehicle that will be making a left turn with some frequency.

2. Special Pavement Markings. As illustrated in Figure 10-49 pavement markings can effectively guide two lines of vehicles turning abreast. See the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) for applicable guidelines on the selection and placement of any special pavement markings.

3. Encroachment. The desirable design will allow the design vehicle to make a left turn and to remain entirely within the through inside lane of the divided facility. In addition, the turning vehicle should be no closer than 600 mm to the inside curb or inside edge of pavement. However, depending on traffic control or available intersection sight distance, it would be acceptable for the design vehicle to occupy both travel lanes; see Figure 10-51.

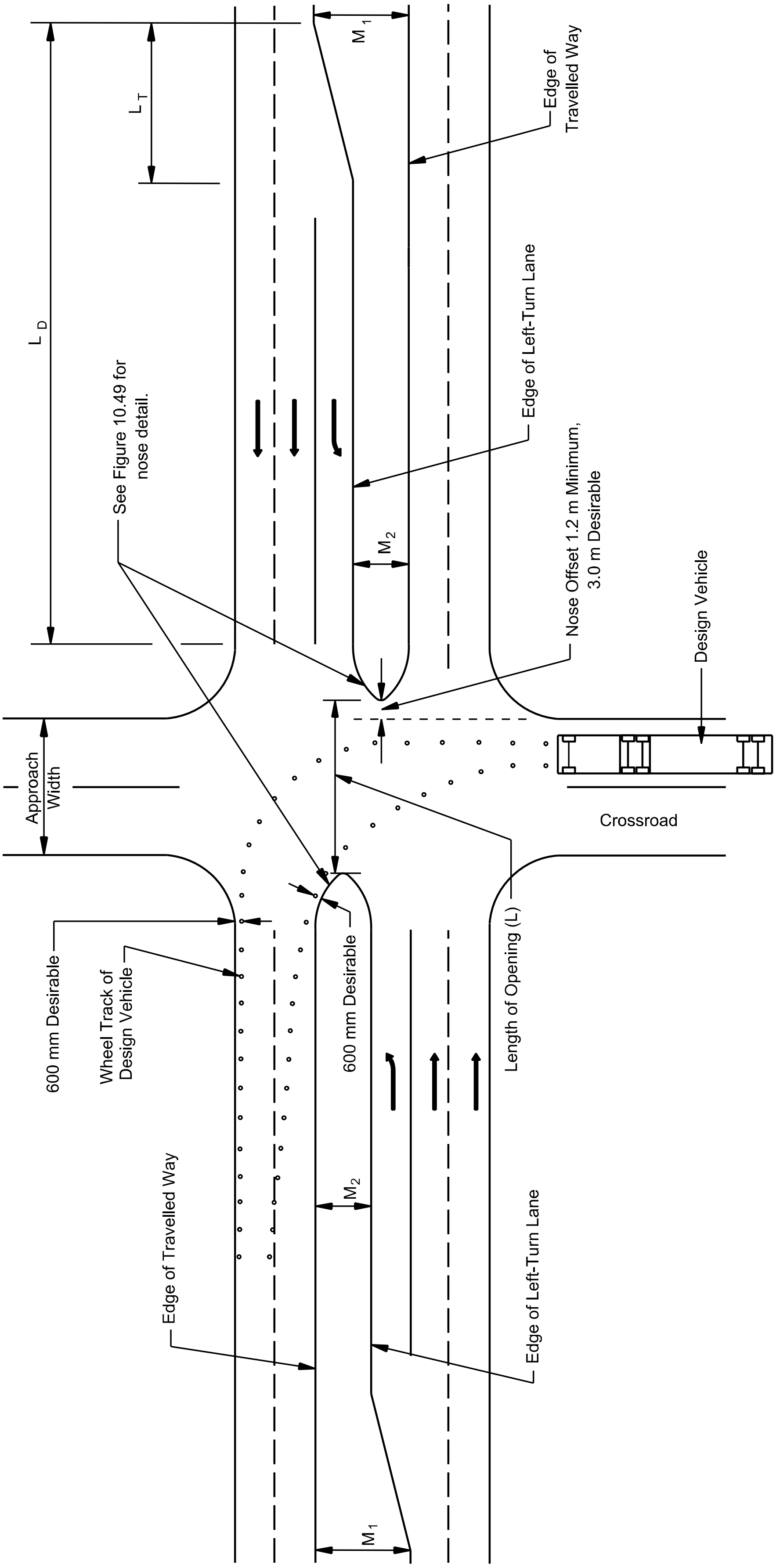
4. Length of Opening. The length of a median opening should properly accommodate the turning path of the design vehicle. The minimum length is the largest of the following:

   - approach width plus 2.4 m, including crossroad median width;
   - approach width plus shoulder widths, including crossroad median width;
   - the length based on the selected design vehicle; or
   - 12 m.

   Table 10-15, Table 10-16, and Table 10-17 provide guidelines for median openings using a P, SU, and WB-12 design vehicles, respectively. Evaluate each median opening individually to determine the proper length. Consider the following factors in the evaluation:

   a. Turning Templates. Check the proposed design with the turning template for the selected design vehicle. Consider the frequency of the turn and encroachment onto adjacent travel lanes or shoulders by the turning vehicle.

Figure 10-51: Median Opening Design



Note: See discussion in Section 10.8.2 for minimum L criteria.

Table 10-15: Minimum Design of Median Openings
(P Design Vehicle,Control Radius of 12m)

Width of Median, M (m)	Minimum Length of Median Opening, L (m)	
	Semicircular	Bullet Nose
1.2	22.8	22.8
1.8	22.2	18.0
2.4	21.6	16.8
3.0	21.0	16.8
3.6	20.4	16.8
4.2	19.8	16.8
4.8	19.2	16.8
6.0	18.0	16.8
7.2	16.8	16.8

Source: (1)

Table 10-16: Minimum Design of Median Openings
(SU Design Vehicle,Control Radius of 15m)

Width of Median, M (m)	Minimum Length of Median Opening, L (m)	
	Semicircular	Bullet Nose
1.2	28.8	28.8
1.8	28.2	22.8
2.4	27.6	20.4
3.0	27.0	18.6
3.6	26.4	17.4
4.2	25.8	16.8
4.8	25.2	16.8
6.0	24.0	16.8
7.2	22.8	16.8
8.4	21.6	16.8
9.6	20.4	16.8
10.8	19.2	16.8
12.0	18.0	16.8

Source: (1)

Table 10-17: Minimum Design of Median Openings (WB-12 Design Vehicle,Control Radius of 23m)

Width of Median, M (m)	Minimum Length of Median Opening, L (m)	
	Semicircular	Bullet Nose
1.2	43.8	36.6
1.8	43.2	36.3
2.4	42.6	33.6
3.0	42.0	31.2
3.6	41.4	29.4
4.2	40.8	27.6
4.8	40.2	26.4
6.0	39.0	23.4
7.2	37.8	21.6
8.4	36.6	19.5
9.6	35.4	18.0
10.8	34.2	16.2
12.0	30.0	14.7
18.0	27.0	13.3
24.0	21.0	13.2
30.0	15.0	13.2

Source: (1)

b. Nose Offset. At four-leg intersections, traffic travelling through the median opening (going straight) will pass the nose of the median end (semicircular or bullet nose). To provide a sense of comfort for these drivers, the offset between the crossroad through travel lane (extended) and the median nose should be at least 1.2 m.

c. Lane Alignment. Provide a design where the lanes line up properly across the intersection.

d. Location of Crosswalks. Desirably, pedestrian crosswalks will intersect the median nose to provide some refuge for pedestrians. Therefore, the median opening design should be coordinated with the location of crosswalks.

e. Traffic Control. Review the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) for signing, striping, and traffic control.

5. Median Nose Design. The shape of the nose at median openings is determined by the width of the median (M_1) or (M_2). The two basic types of median nose designs are the semicircular design and bullet-nose design. The following summarises their usage:

- For medians up to 1.2 m in width, there is little operational difference between the two designs.
- The semicircular design is generally acceptable for median widths (M_1) up to 3.0 m.

- For medians (M_1) wider than 3.0 m, use the bullet-nose design. Also, use this design for the divisional island remaining after locating a left-turn lane in median.
- As medians become successively wider, the minimum length of the median opening becomes the governing design control.
- For the bullet-nose design, a compound curvature arrangement should be used. Figure 10-52 provides the typical details for a median opening with a bullet-nose design.

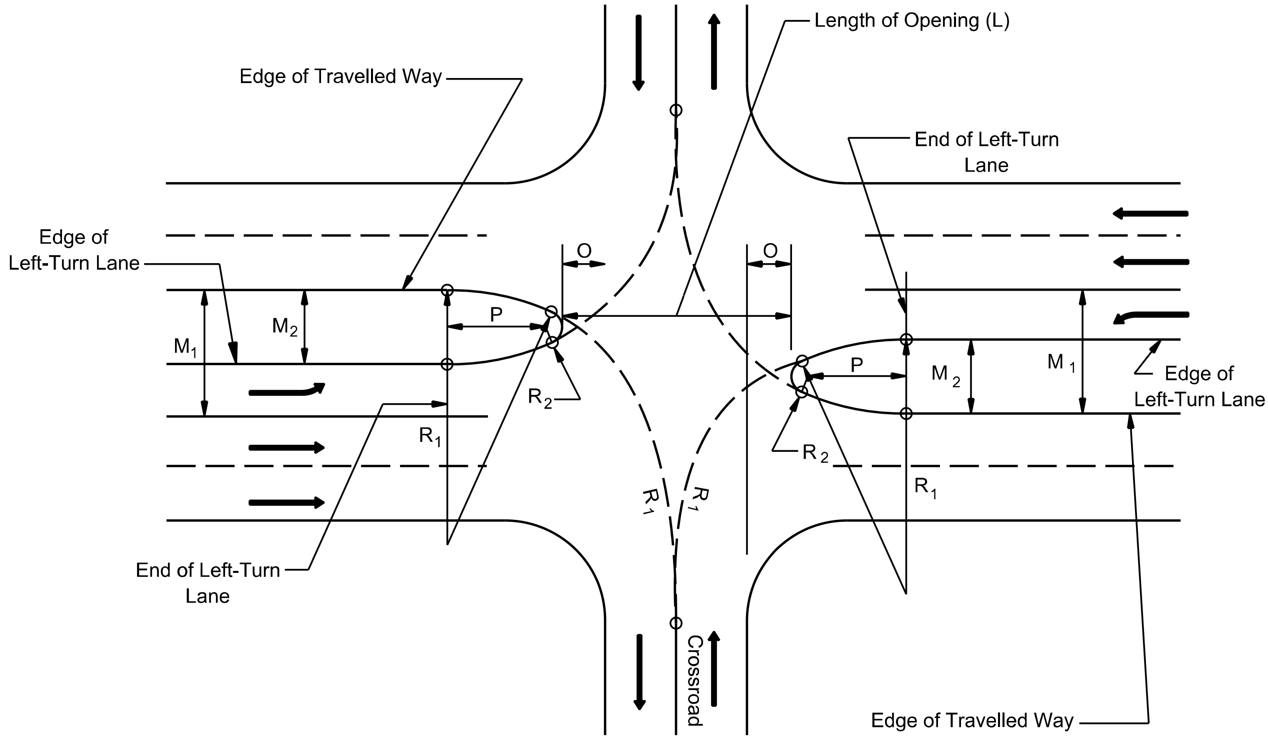
6. U-Turns. Median openings are sometimes used to accommodate U-turns on multilane divided roadways; see Figure 10-53 and Figure 10-54. Preferably, a vehicle should be able to begin and end the U-turn on the inner lanes next to the median. See Section 10.8.3.
7. Sight Distance. Check all median openings for applicable sight distance criteria.
8. Effect of Skew. Table 10-18 provides preliminary design guidance for skewed median crossings. Table 10-18 uses an assumed control radius of 15 m. Median openings are measured normal to the crossroad. For the actual design, each skewed median opening must be design individually using the guidance provided in Section 10.8.2.

10.8.3 Minimum Design for U-Turns

U-turns enhance motor vehicle traffic flow, facilitate access management, and reduce left turn pressure at junctions. Table 10-19 provides the minimum median widths for rural U-turn manoeuvres for various design vehicles and various levels of encroachment. An important factor in designing median openings is the path of each design vehicle making a U-turn at 15 km/h to 25 km/h. Where the volume and type of vehicles making the left-turn movement call for higher than minimum speed, the design may be made by using a radius of turn corresponding to the speed deemed appropriate. However, the minimum turning path at low speed is needed for the minimum design and for testing layouts developed for one design vehicle for use by an occasional larger vehicle. Check all U-turn design with the applicable turning template and ensure the design can accommodate the swept path width for the occasional larger vehicle without having the truck back up or having the rear trailer wheels encroach upon curbs, signs, and pedestrian waiting areas.

In urban areas, U-turns may be used on Boulevards and Avenues. Adjust the values from Table 10-9 using urban lane and median widths. Locate the U-turn before the crosswalk. This eliminates turning vehicles conflicts with crossing pedestrians. Signalise the U-turn opening where there are two or more receiving lanes. On Avenues, if required, provide local bulbing for large vehicles to safely complete U-turns as shown in the bottom figure of Table 10-19. This will prevent turning vehicles from having to back up while making U-turns.

Figure 10-52: Median Nose Design



R_1 = variable, based on design vehicle and median width (M_2)

R_2 = $M_2/5$ to edge of left-turn lane

R_2 = $M_1/5$ to edge of travelled way where a left-turn lane is not present
R_2 is typically rounded up to the next highest whole number

R_1 = variable, based on design vehicle and median width (M_2)

R_2 = $M_2/5$ to edge of left-turn lane

R_2 = $M_1/5$ to edge of travelled way where a left-turn lane is not present
R_2 is typically rounded up to the next highest whole number

L = Length of median opening. See discussion in Section 10.8.2 for minimum L values.

M_1 = median width measured between the two edges of the inside travel lanes

M_2 = width of divisional island (raised or depressed) remaining after the width of the left-turn (if present) has been subtracted from the median width (M_1)

O = Nose offset.

P = As shown in figure.

Figure 10-53: Urban Median U-Turns

Figure 10-54: Rural Median U-Turns

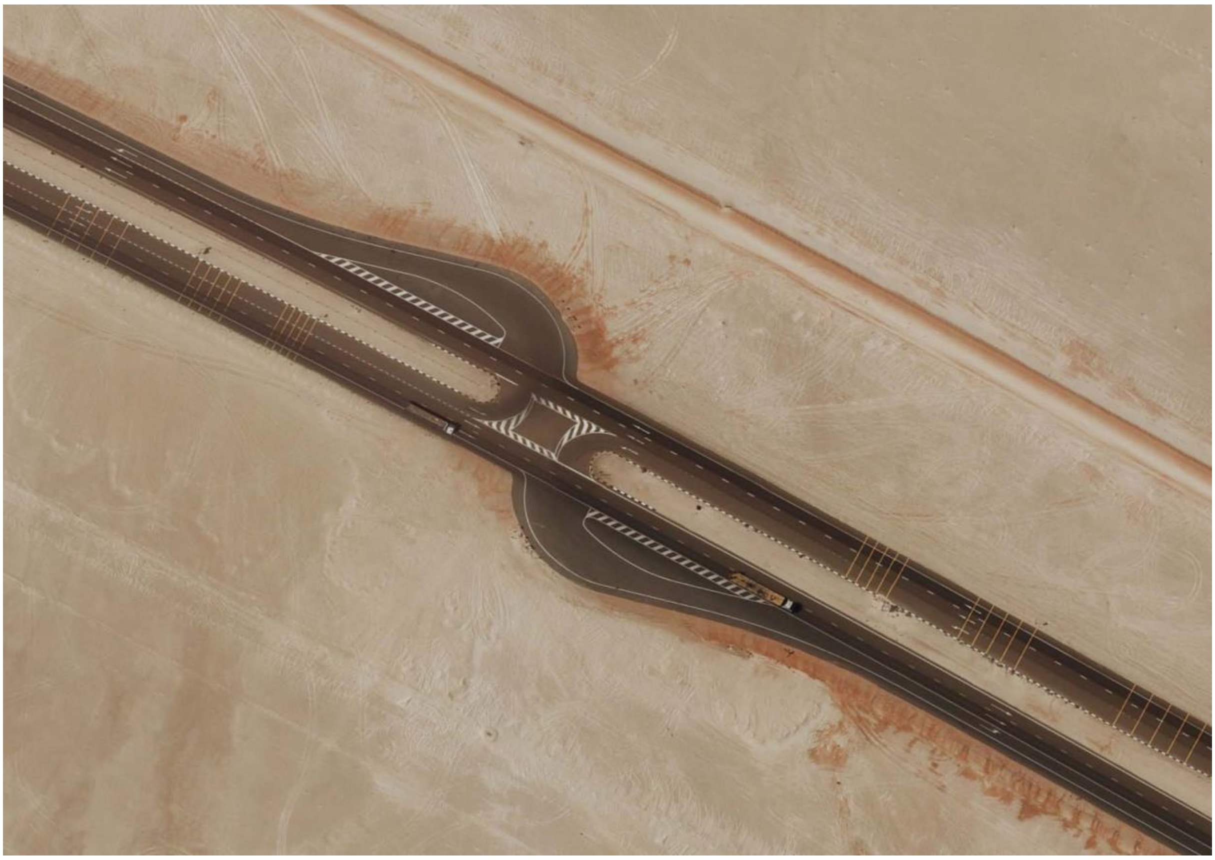
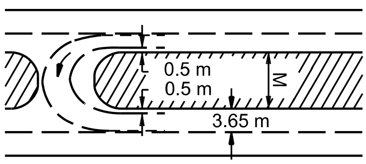
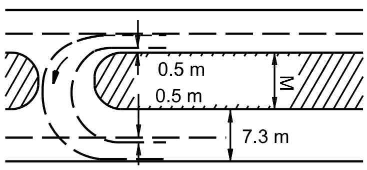
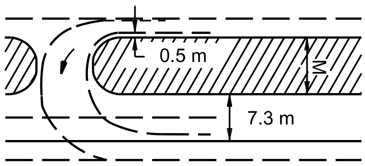
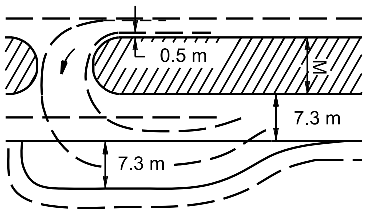


Table 10-18: Preliminary Design Guidance for Skewed Median Openings

Skew Angle from 90˚ (°)	Width of Median (m)	Semi-Circular Nose	Length of Median Opening Measured Normal to the Crossroad (m) Bullet Nose		Radius (m)
			Symmetrical Bullet Nose	Asymmetrical Bullet Nose	
0	3	27	19	-	-
	6	24	13	-	-
	9	21	16.8 min	-	-
	12	18	16.8 min	-	-
10	3	32	24	23	21
	6	28	17	16.8 min	20
	9	25	16.8 min	16.8 min	20
	12	21	16.8 min	16.8 min	19
	15	18	-	-	-
20	3	39	30	-	29
	6	35	24	20	28
	9	31	19	14	26
	12	26	17	12 min	25
	15	23	17	12 min	23
	18	18	-	-	-
30	3	48	40	32	42
	6	43	32	23	39
	9	38	26	17	36
	12	34	22	13	33
	15	27	18	12 min	30
	18	24	15 min	12 min	27
40	3	60	52	35	63
	6	55	43	27	58
	9	49	37	20	53
	12	43	30	15	47
	15	37	26	12 min	42
	18	32	23	12 min	36

Source: (1)

Table 10-19: Minimum Widths Needed for U-Turns

Type of Manoeuvre		M - Min. Width of Median for Design Vehicle				
		P	SU	BUS	WB-12	WB-15
		Length of Design Vehicle				
		5.7 m	9.0 m	12.0 m	15.0 m	16.5 m
Inner Lane to Inner Lane		9 m	19 m	19 m	18 m	21 m
Inner Lane to Outer Lane		5 m	15 m	15 m	15 m	18 m
Inner Lane to Shoulder		2 m	12 m	12 m	12 m	15 m
Inner Lane to Jug Handle		2 m	10 m	10 m	10 m	12 m

Note: In urban areas, adjust the median widths using urban lane widths.

Source: (1) *revised*

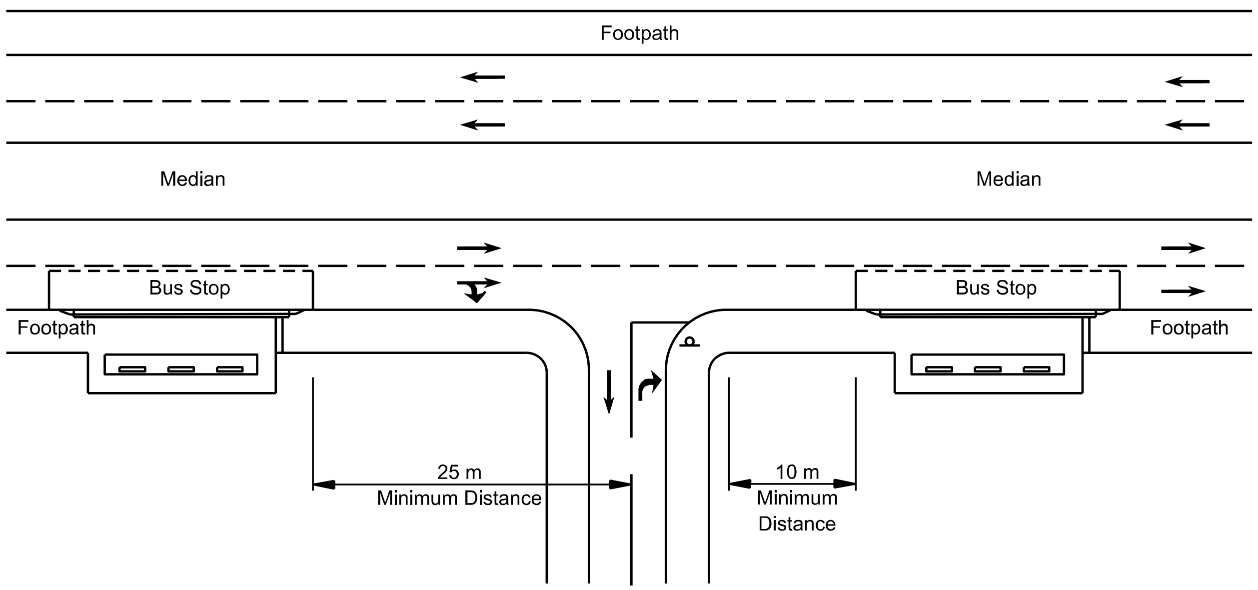
Figure 10-55: U-Turns at Intersections

10.9 Placement of Bus Stops Near Intersections

When locating bus stops near intersections, consider the following guidance:

1. Pedestrian crossings should be located before vehicle waiting areas or bus stops to avoid conflicts and delays.
2. Avoid locating transit stops adjacent to curb cuts, driveways, and busy vehicular entrances to minimise vehicular conflicts.
3. Where the bus stop will compromise the pedestrian realm or cycle facilities, eliminate parking on the frontage lane, and/or narrow the combined width of the Edge and Furnishings zones.
4. Provide a minimum 2-m obstacle-free clear zone around transit stops to ensure accessibility.
5. Figure 10-56, Figure 10-57, and Figure 10-58 provide design guidance in locating bus stop location near intersections. Figure 10-59 shows a bus stop after an intersection.

Figure 10-56: Curb Side Bus Stops Near Right-In, Right-Out Junction



Source: (49)

Figure 10-57: Curb Side Bus Stops Near Roundabout

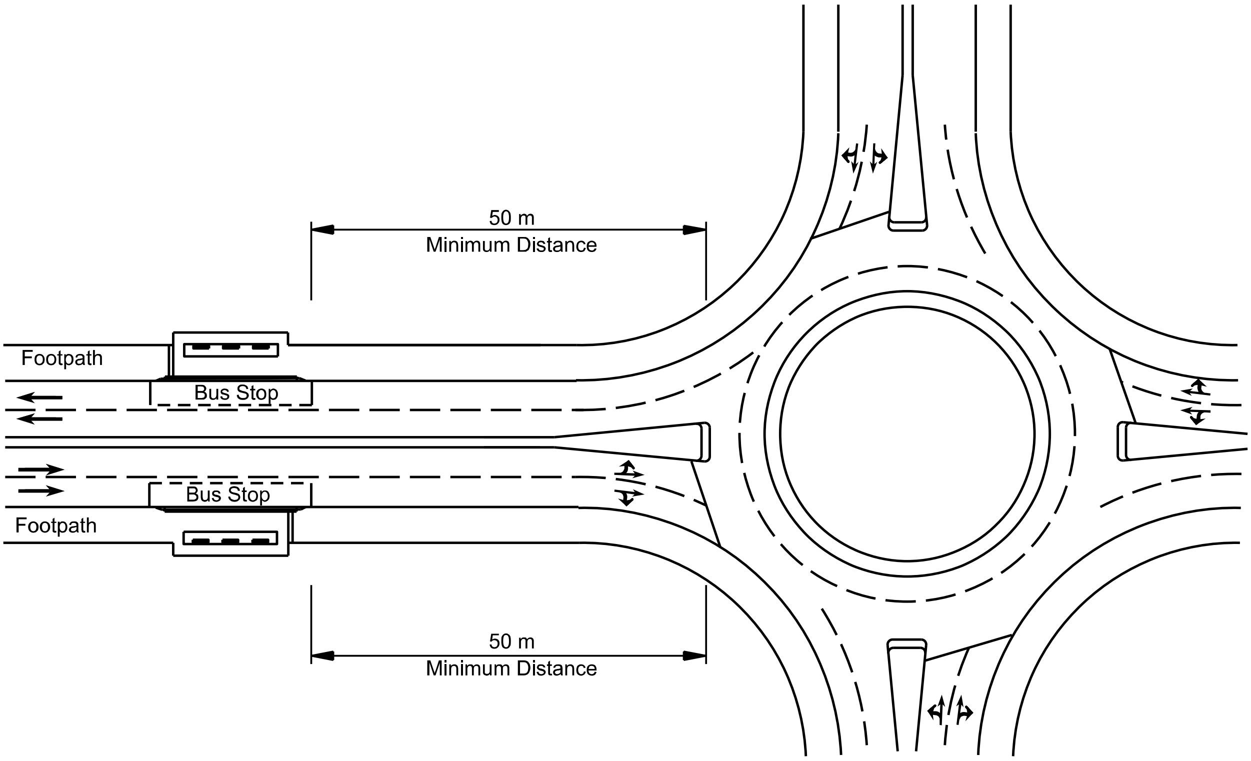


Figure 10-58: Curb Side Bus Stops at Urban Signal Junction

Table 1

Distance "A" (m)	Approach Type
150	Major
90	Minor

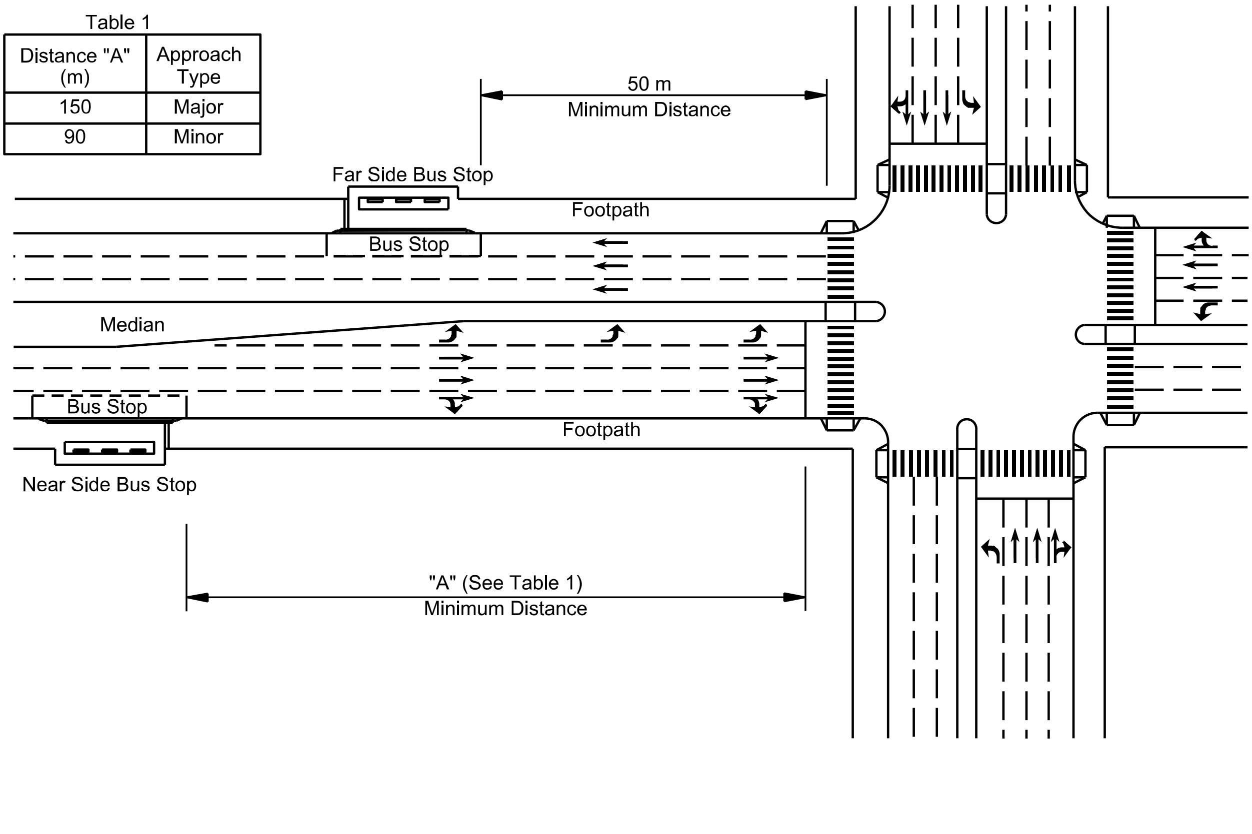


Figure 10-59: Far Side Bus Stop

10.10 Intersections with Service Roads

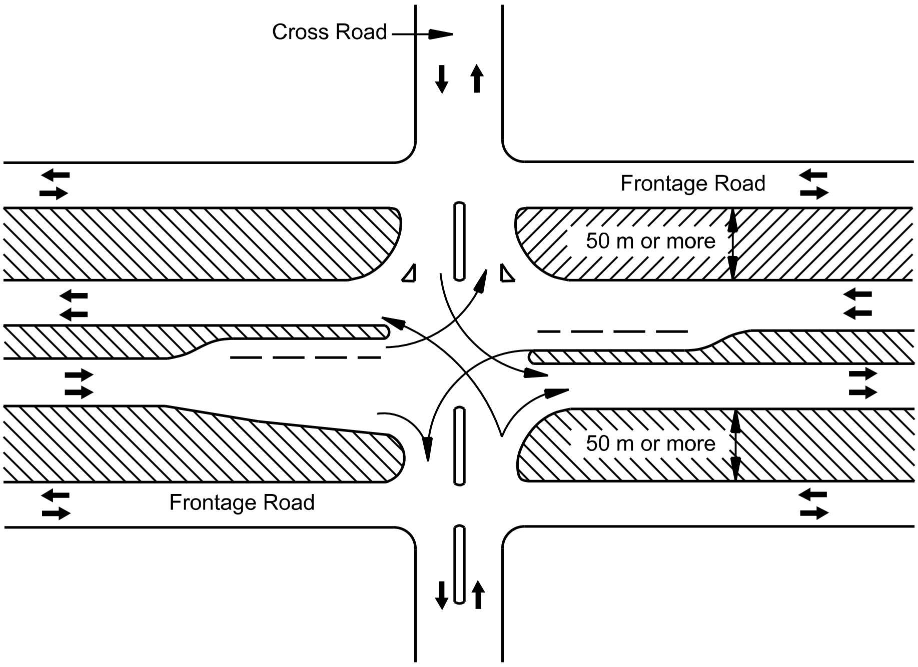
Intersections with frontage/service roads typically have two or three separate intersections within a short distance; see Figure 10-60. These short separations may introduce added conflicts, reduced capacity, longer signal times, cause confusion leading to wrong-way entries, cause problems for u-turning vehicles, etc. Desirably, the frontage road should be set back 50 m or more from the main intersection. Outer separations of 100 m or more will enhance operations, allow for overlapping left-turn lanes, and provide a minimal amount of vehicle storage between the intersections. The design year traffic volumes, turning movements, signal phasing, and storage needs should determine the ultimate separation distance.

Narrow separations are acceptable where the frontage road volumes are light, the frontage road is a one-way, or where some movements can be restricted. Turning movements that are affected most by the width of the outer separation are:

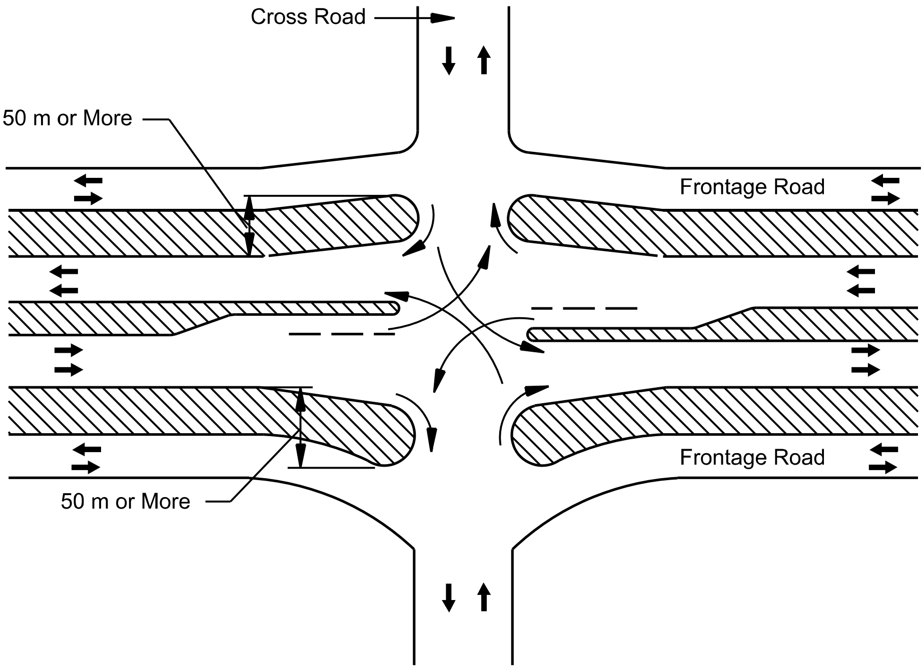
- left turns from the frontage road onto the crossroad,
- U-turns from the through lanes of the main road onto a two-way frontage road, and
- right turns from the through lanes of the main road onto the crossroad.

With one or more of these restrictions, outer separations of 2.4 m may operate successfully. However, the designer needs to assess the risk for wrong way entries onto the mainline.

Figure 10-60: Intersections with Frontage Roads



(a) Two-Way Frontage Roads,
Wide Outer Separation



(b) Two-Way Frontage Roads,
Bulbed Separation

Except for the width of the outer separation, the design elements for intersections involving frontage roads are much the same as those for conventional intersections. Figure 10-60 shows two arrangements of roadways with frontage roads intersecting cross streets. Turning movements are shown on the assumption that frontage road volumes are very light and that all movements will be under traffic signal control. Figure 10-61 shows an intersection with frontage roads.

Figure 10-61: Intersection with Frontage Roads

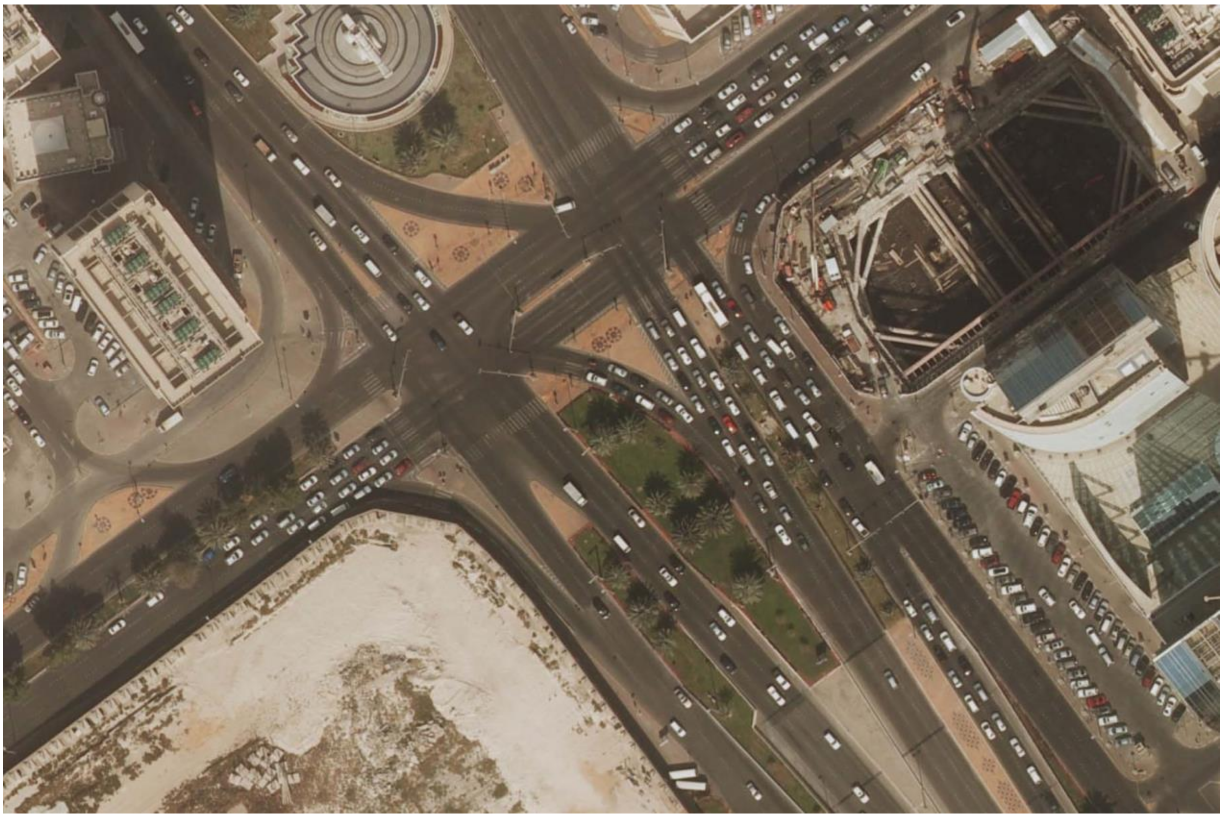


Figure 10-60 (a) shows a simple intersection with an outer separation of 50 m or more in width. The intersections of the two-way frontage roads and the crossroad are sufficiently removed from the through roadways that they might operate as separate intersections. The major elements in the design of the outer intersections are adequate width, adequate radii for right turns, and divisional islands on the crossroad.

Figure 10-60(b) shows a design that would be adaptable for two-way frontage roads in areas where right-of-way considerations would preclude the design shown in Figure 10-60(a). With narrow outer separations between intersections, a bulb treatment of the outer separations, as shown, formed by a reverse-curve alignment of the frontage road on each side of the crossroad is needed to widen the outer separation to a desirable width at the crossroad. The length of the reverse curve is a matter of frontage road design, governed by design speeds and right-of-way controls. The widths of the outer separation bulbs should be based on the pattern and volumes of traffic, but the right-of-way controls may govern because additional area is needed at the intersection. The width of the outer separation should be at least 18 m, but preferably be 50 m or more. The minimum median widths shown in Table 10-19 for median U-turns are also applicable for vehicles making U-turns from the mainline onto the frontage road.

11 ROUNDABOUTS

11.1 Overview

This chapter discusses the various types of runabouts – single lane roundabouts, multilane roundabouts, and mini-roundabouts. Design guidance is provided on the various design elements of the roundabout. For additional guidance on roundabouts, the designer should review the AASHTO *A Policy on Geometric Design of Highways and Streets* (1) and NCHRP Report 672 *Roundabouts: An Informational Guide, Second Edition* (50). As an alternative to the criteria in this *Manual*, the designer may consider the design criteria provided in the Highways Agency of Scotland Transport *Geometric Design of Roundabouts Manual* (51).

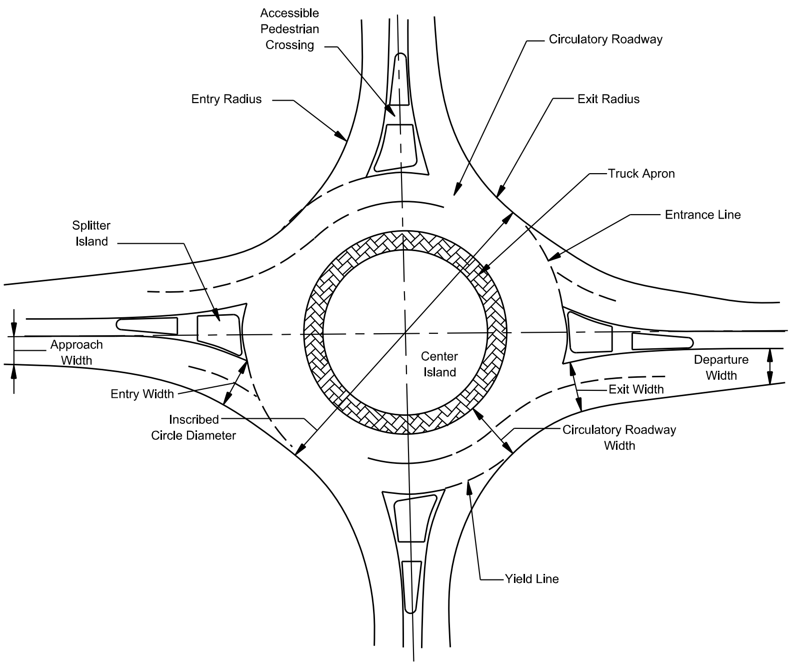
11.2 Characteristics of a Roundabout

11.2.1 Design Features

A roundabout is a form of circular intersection in which traffic travels counter clockwise (in Abu Dhabi and other right-hand traffic countries) around a central island and in which entering traffic must yield to circulating traffic. illustrates a typical roundabout, annotated to identify the key characteristics.

Table 11.1 provides a description of each of the key features.

Figure 11-1: Roundabout Features



Source: (50)

Table 11-1: Roundabout Features

Feature	Description
Central Island	The central island is the raised area in the centre of a roundabout around which traffic circulates. The central island does not necessarily need to be circular in shape. In the case of mini-roundabouts, the central island is traversable.
Splitter Island	A splitter island is a raised or painted area on an approach used to separate entering from exiting traffic, deflect and slow entering traffic, and allow pedestrians to cross the road in two stages.
Circulatory Roadway	The circulatory roadway is the curved path used by vehicles to travel in a counter clockwise fashion around the central island.
Truck Apron	An apron is the traversable portion of the central island adjacent to the circulatory roadway that may be needed to accommodate the wheel tracking of large vehicles. An apron is sometimes provided on the outside of the circulatory roadway
Entrance Line	The entrance line marks the point of entry into the circulatory roadway. This line is physically an extension of the circulatory roadway edge line but functions as a yield or give-way line in the absence of a separate yield line. Entering vehicles must yield to any circulating traffic coming from the left before crossing this line into the circulatory roadway.
Accessible Pedestrian Crossings	For roundabouts designed with pedestrian pathways, the crossing location is typically set back from the entrance line, and the splitter island is typically cut to allow pedestrians, wheelchairs, strollers, and cyclists to pass through. The pedestrian crossings must be accessible with detectable warnings and appropriate slopes.
Landscape Strip	Landscape strips separate vehicular and pedestrian traffic and assist with guiding pedestrians to the designated crossing locations. This feature is particularly important as a wayfinding cue for individuals who are visually impaired. Landscape strips can also significantly improve the aesthetics of the intersection.

Source: (50)

Roundabouts often include one or more additional design features intended to enhance the safety and/or capacity of the intersection. However, their absence does not necessarily preclude an intersection from operating as a roundabout. These additional features include:

1. Entry Flare. Flare on an entry to a roundabout is the widening of an approach to multiple lanes to provide additional capacity and storage at the entrance line.

2. Splitter Island. All but some mini-roundabouts have raised splitter islands. These are designed to separate traffic moving in opposite directions, deflect entering traffic, and to provide opportunities for pedestrians to cross in two stages. Mini-roundabouts may have splitter islands defined only by pavement markings.

3. Pedestrian Crossing Locations. Pedestrian crossings are located only across the legs of the roundabout, typically separated from the circulatory roadway by at least one vehicle length.

4. Parking. No parking is allowed within the circulatory roadway or at the entries. Parking manoeuvres within the intersection, as is the case at some traffic circles, interfere with circulatory flow and present a potential safety hazards.

11.2.2 Roundabouts Characteristics

Table 11-2 identifies some of the major characteristics of roundabouts.

Table 11-2: Roundabout Characteristics

Roundabouts
Traffic Control
Yield control is used on all entries. The circulatory roadway has no control.
Priority to Circulating Vehicles
Circulating vehicles have the right-of-way.
Direction of Circulation
All vehicles circulate counter clockwise and pass to the right of the central island.
Adequate Speed Reduction
Good roundabout design requires entering vehicles to negotiate the roundabout at slow speeds. Once within the circulatory roadway, vehicle paths are further deflected by the central island.
Design Vehicle
Good roundabout design makes accommodation for the appropriate design vehicle. This may require the use of an apron.

Source: (50)

11.3 Categories of Roundabouts

For design purposes, roundabouts are separated into three basic categories according to size and number of lanes — mini-roundabouts, single-lane roundabouts, and multilane roundabouts.

Note that separate categories have not been explicitly identified for rural and urban areas. Roundabouts in urban areas may require smaller inscribed circle diameters due to smaller design vehicles and existing right-of-way constraints. They may also include more extensive pedestrian and cyclists features. Roundabouts in rural areas typically have higher approach speeds and may require special attention to visibility, approach alignment, and cross-sectional details.

Table 11-3 summarises and compares some fundamental design and operational elements for each of the three roundabout categories. The following sections provide a qualitative discussion of each category. Figure 11-2 illustrates a roundabout at an intersection, Figure 11-3 shows single-lane roundabout, Figure 11-4 shows a multilane roundabout, and shows a roundabout interchange.

Table 11-3: Roundabout Category Comparison

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Desirable maximum entry design speed	25 to 30 km/h	30 to 40 km/h	40 to 50 km/h
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	13 m to 27 m	27 m to 55 m	46 m to 91 m
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)
Typical daily service volumes on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)*	Up to approximately 15,000	Up to approximately 25,000	Up to approximately 45,000 for two-lane roundabout

* *Operational analysis needed to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.*

Source: (50)

Figure 11-2: Roundabout for Freeway Ramp and Crossroad

Figure 11-3: Single Lane Roundabout

Figure 11-4: Multilane Roundabout (Sharjah)

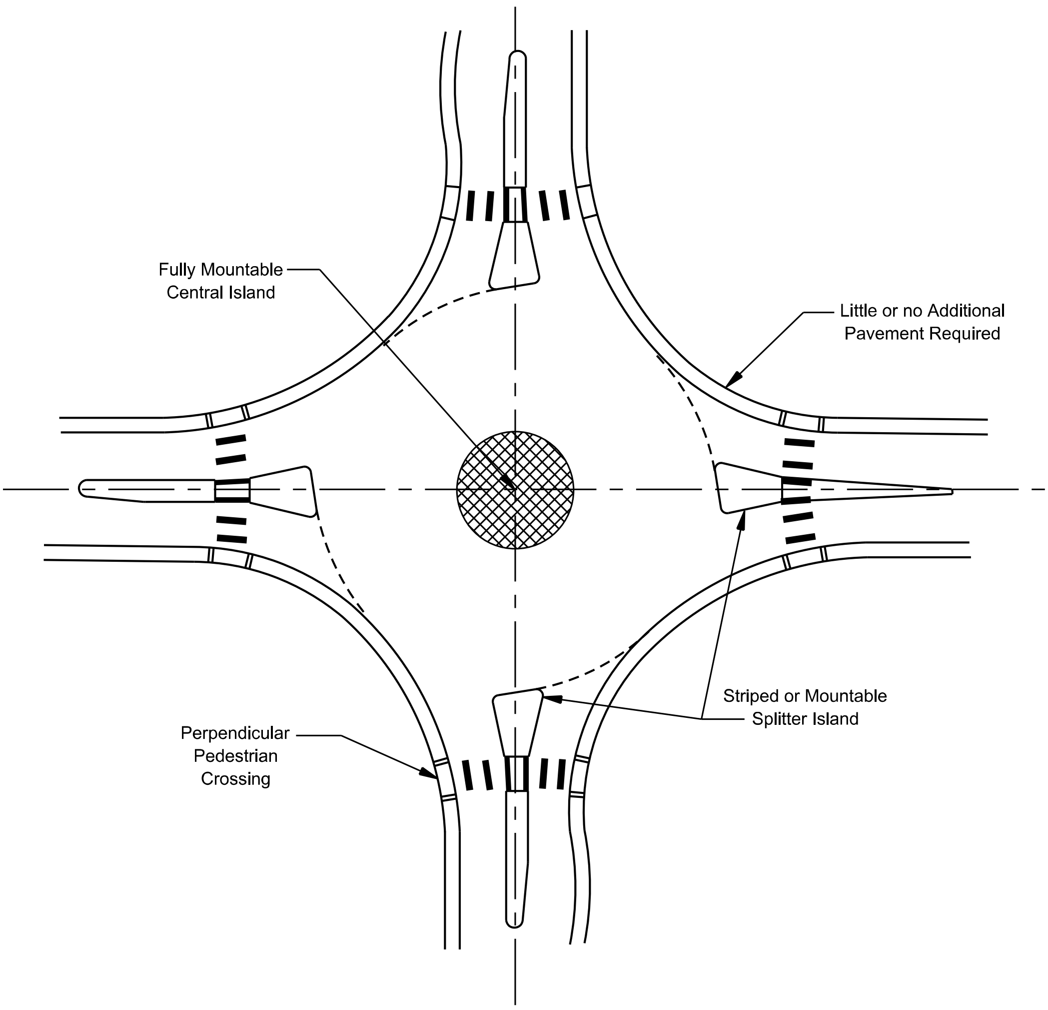
Figure 11-5: Roundabout Interchanges

11.3.1 Mini-Roundabouts

Mini-roundabouts are small roundabouts with a fully traversable central island. They are most commonly used in low-speed urban environments with average operating speeds of 50 km/h or less. Figure 11-6 shows the features of typical mini-roundabouts. They may be used where conventional roundabout design is precluded by right-of-way constraints. In retrofit applications, mini-roundabouts are relatively inexpensive because they typically require minimal additional pavement at the intersecting roads and minor widening at the corner curbs. They are mostly used where there is insufficient right-of-way to accommodate the design vehicle with a traditional single-lane roundabout. Because they are small, mini-roundabouts are perceived as pedestrian-friendly with short crossing distances and very low vehicle speeds on approaches and exits.

A fully traversable central island is provided to accommodate large vehicles and serves one of the distinguishing features of a mini-roundabout. The mini-roundabout is designed to accommodate passenger cars without requiring them to traverse over the central island. The overall design of a mini-roundabout should align vehicles at entry to guide drivers to the intended path and minimise running over of the central island to the extent practical.

Figure 11-6: Features of Typical Mini-Roundabout



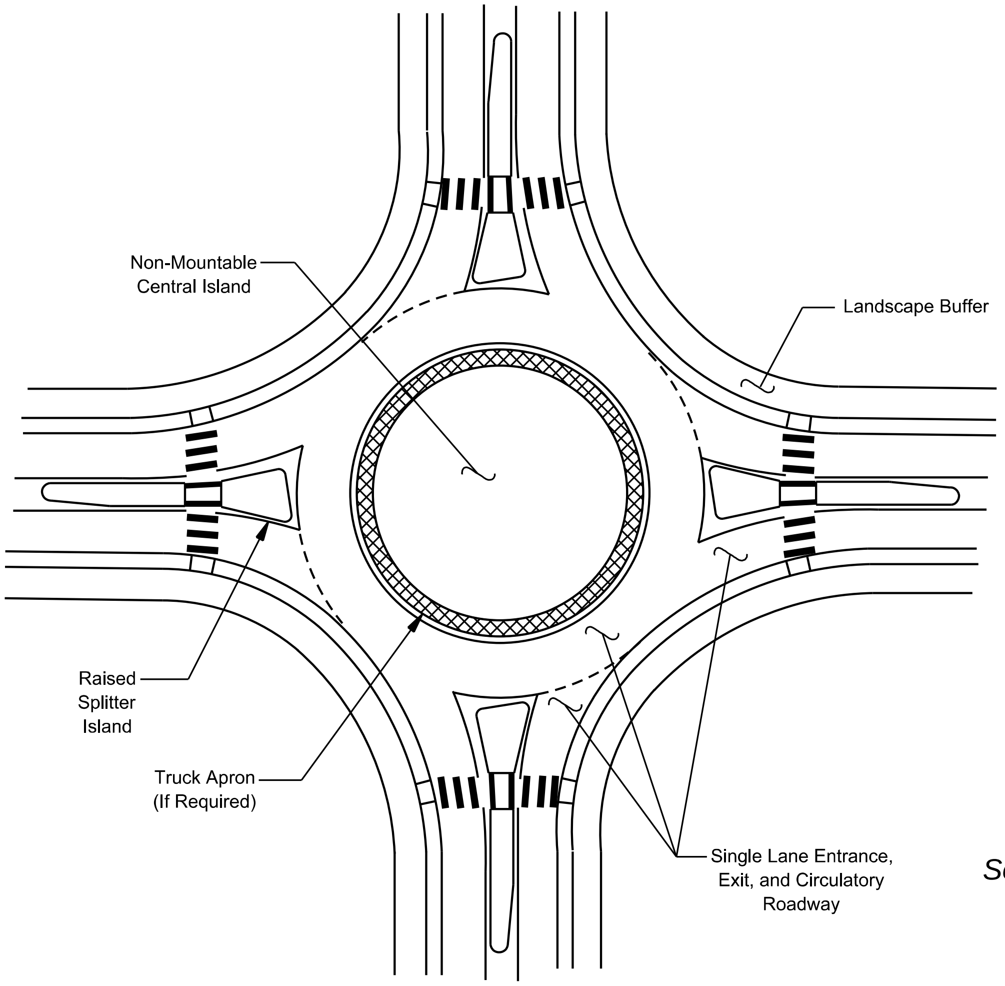
11.3.2 Single-Lane Roundabouts

This type of roundabout is characterised as having a single-lane entry at all legs and one circulatory lane. Figure 11-7 shows the features of typical single-lane roundabouts. They are distinguished from mini-roundabouts by their larger inscribed circle diameters and non-traversable central islands. Single-lane roundabouts design allows slightly higher speeds at the entry, on the circulatory roadway, and at the exit. The geometric design typically includes raised splitter islands, a non-traversable central island, crosswalks, and a truck apron. The size of the roundabout is largely influenced by the applicable design vehicle and available right-of-way.

11.3.3 Multilane Roundabouts

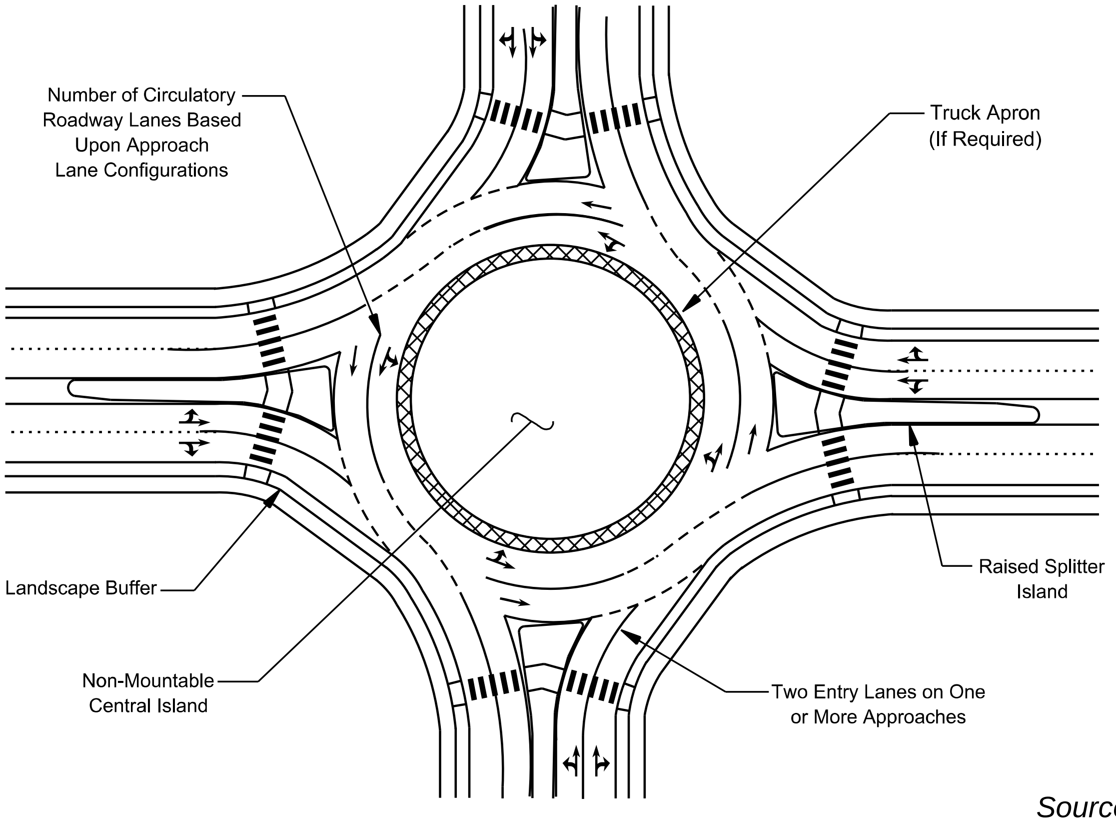
Multilane roundabouts have at least one entry with two or more lanes. In some cases, the roundabout may have a different number of lanes on one or more approaches (e.g. two-lane entries on the major street and one-lane entries on the minor street). They also include roundabouts with entries on one or more approaches that flare from one to two or more lanes. These require wider circulatory roadways to accommodate more than one vehicle traveling side by side. Figure 11-8, Figure 11-9, and Figure 11-10 provide examples of typical multilane roundabouts. The speeds at the entry, on the circulatory roadway, and at the exit are similar or may be slightly higher than those for the single-lane roundabouts. The geometric design includes raised splitter islands, truck apron, a non-traversable central island, and appropriate entry path deflection.

Figure 11-7: Features of Typical Single-Lane Roundabout



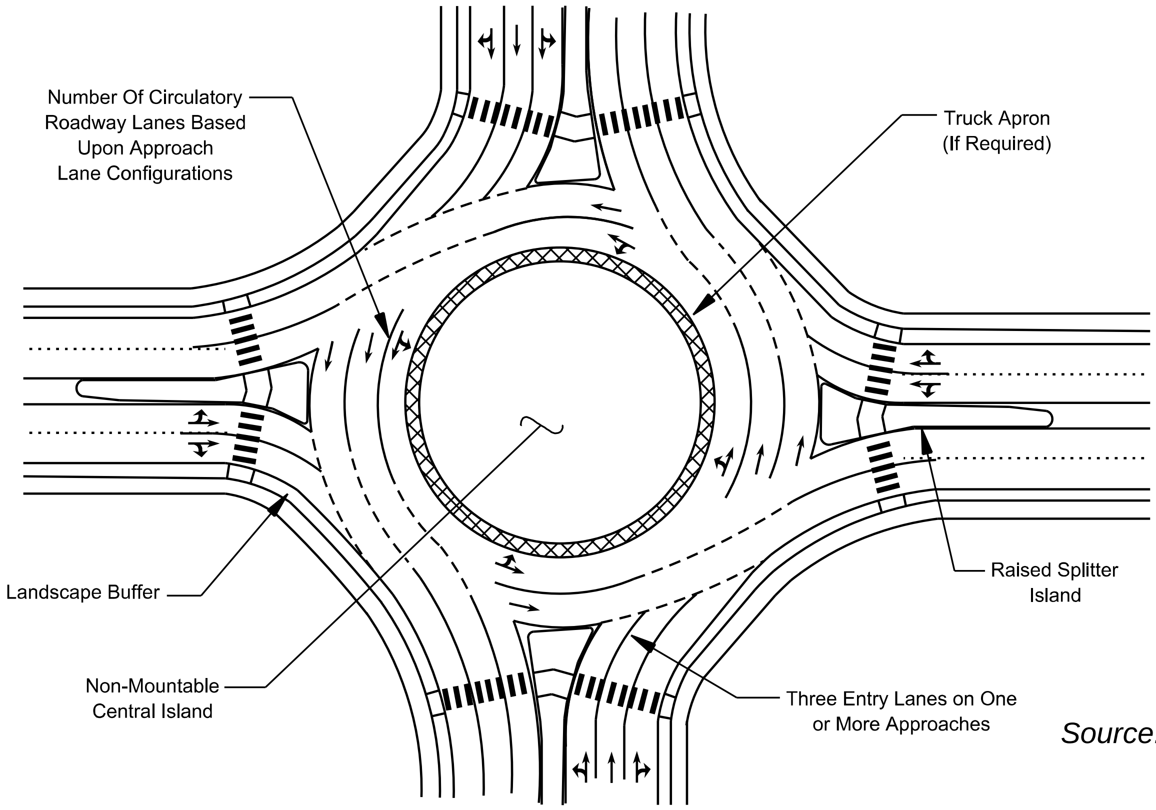
Source: (50)

Figure 11-8: Features of Typical Two-Lane Roundabout



Source: (50)

Figure 11-9: Features of Typical Three-Lane Roundabout



Source: (50)

Figure 11-10: Multilane Roundabout

11.4 Roundabout Considerations

11.4.1 Planning

At the planning stage, there are a variety of possible reasons or goals for considering a roundabout at a particular intersection. These may include:

- improving safety or operations,
- improving aesthetics,
- assisting with access management, and
- promoting redevelopment.

Whatever the reasons for considering a roundabout, a number of common considerations should be addressed at the planning level:

- Is a roundabout appropriate for this location?
- How big should it be or how many lanes might be required?
- What sort of impacts might be expected?
- What public education and outreach might be appropriate?

11.4.2 General

Roundabouts features and characteristics can affect roundabout characteristics safety, signal progression, environmental factors, spatial requirements, operation and maintenance costs, traffic calming, aesthetics, and access management. The trade-offs involved when implementing a roundabout should be considered at a policy level when introducing roundabouts into a region or on a project-by-project basis at specific locations where a roundabout is one of the alternatives being considered.

Table 11-4 provides an overview of the primary advantages and disadvantages of roundabouts when considering this type of intersection. Note that roundabouts with more than two lanes are not desirable in urban areas.

11.4.3 User Considerations

As with any intersection design, each expected user requires careful consideration during the design. The designer should consider the following guidelines:

1. Pedestrians. Pedestrians are accommodated at crosswalks around the perimeter of the roundabout. By providing space to pause on the splitter island, pedestrians can consider one direction of conflicting traffic at a time, which simplifies the task of crossing the street. Design the roundabout to discourage pedestrians from crossing to the central island. Roundabouts may cause confusion for blind pedestrians, because crosswalks are located outside the projection of approaching sidewalks, and the curvilinear nature of roundabouts alters the normal audible and tactile cues they use to find crosswalks.

2. Cyclists. One of the difficulties in accommodating cyclists is their wide range of skills and comfort levels in mixed traffic. Some of the least-skilled cyclists will choose to ride on sidewalks both along streets away from roundabouts and at the roundabouts. Because these cyclists are behaving like rolling pedestrians, no specific treatments are necessary at roundabouts besides what are provided for pedestrians. In general, cyclists who have the knowledge and skills to ride effectively and safely on roadways can navigate low-speed single lane roundabouts without much difficulty. See the ABU DHABI *Walking and Cycling Master Plan* (23) for accommodations of cyclists at roundabouts.

3. Elderly Drivers. Driving situations involving complex speed–distance judgments under time constraints are more problematic for older drivers. One of the key design features of a roundabout is that all traffic must slow down as it enters the intersection. Slower speeds can benefit elderly drivers as they navigate the roadways.

4. Large Vehicles. Large vehicles have a direct impact on the design of a roundabout. Single-lane roundabouts often employ a traversable apron around the perimeter of the central island to provide the additional width needed for tracking the trailer wheels of large vehicles. Multilane roundabouts are designed either to allow large vehicles to track across more than one lane while entering, circulating, and exiting or to stay within their lane.

Table 11-4: Summary of Roundabout Advantages and Disadvantages

Advantages	Disadvantages
Non-Motorised Users	
• Pedestrians must consider only one direction of conflicting traffic at a time. • Bicyclists have options for negotiating roundabouts, depending on their skill and comfort level.	• Pedestrians with vision impairments may have trouble finding crosswalks and determining when/if vehicles have yielded at crosswalks. • Cycle ramps at roundabouts have the potential to be confused with pedestrian ramps.
Safety	
• Reduce crash severity for all users, allow safer merges into circulating traffic, and provide more time for all users to detect and correct for their mistakes or the mistakes of others due to lower vehicle speeds. • Fewer overall conflict points and no left-turn conflicts. • Lower rates of fatal and serious crashes	• Increase in low-severity single-vehicle and fixed-object crashes compared to other intersection treatments. • Multilane roundabouts present more difficulties for individuals with blindness or low vision due to challenges in detecting gaps and determining that vehicles have yielded at crosswalks.
Operations	
• May have lower delays and queues than other forms of intersection control. • Can reduce lane requirements between intersections, including bridges between interchange ramp terminals. • Creates possibility for adjacent signals to operate with more efficient cycle lengths where the roundabout replaces a signal that is setting the controlling cycle length.	• Equal priority for all approaches can reduce the progression for high volume approaches. • Cannot provide explicit priority to specific users (e.g. trains, emergency vehicles, transit, pedestrians) unless supplemental traffic control devices are provided.
Access Management	
• Facilitate U-turns that can substitute for more difficult midblock left turns.	• May reduce the number of available gaps for midblock unsignalised intersections and driveways.
Environmental Factors	
• Noise, air quality impacts, and fuel consumption may be reduced. • Little stopping during off-peak periods	• Possible impacts to natural and cultural resources due to greater spatial requirements at intersections.
Traffic Calming	
• Reduced vehicular speeds. • Beneficial in transition areas by reinforcing the notion of a significant change in the driving environment.	• More expensive than other traffic calming treatments.
Space	
• Often require less queue storage space on intersection approaches—can allow for closer intersection and access spacing. • Reduce the need for additional right-of-way between links of intersection. • More feasibility to accommodate parking, wider sidewalks, planter strips, wider outside lanes, and/or cycle lanes on the approaches.	• Often requires more space at the intersection itself than other intersection treatments.
Operation and Maintenance	
• No signal hardware or equipment maintenance.	• May require landscape maintenance.
Aesthetics	
• Provide attractive entries or centrepieces to communities. • Used in tourist or shopping areas to separate commercial uses from residential areas. • Provide opportunity for landscaping and/or gateway feature to enhance the community.	• May create a safety hazard if hard objects are placed in the central island directly facing the entries.

Source: (50)

5. Transit. If the roundabout has been designed using the appropriate design vehicle, a bus should have no physical difficulty negotiating the intersection.

6. Emergency Vehicles. The passage of large emergency vehicles through a roundabout is the same as for other large vehicles and may require use of a traversable apron. On emergency response routes, the delay for the relevant movements at a planned roundabout should be compared with alternative intersection types and control.

11.4.4 Access Management

Roundabouts can be used at key public and private intersections to facilitate major movements and enhance access management. Minor public and private access points between roundabouts can be accommodated by partially or fully restricted two-way stop-controlled intersections, with the roundabouts providing U-turn opportunities. Most access management principles used for at conventional intersections also applied to roundabouts.

While roundabouts may allow for fewer lanes between intersections, the traffic pattern that emerges from roundabouts can have a significant impact on existing midblock access. The more random departure pattern that emerges from a roundabout and the potentially narrower cross section between roundabouts may reduce the number of available gaps for mid-block unsignalised intersections and driveways. As a result, an unsignalised intersection may have less capacity and more delay downstream of a roundabout than downstream of a signal, even accounting for the U-turns that roundabouts facilitate. This should be reviewed on a case-by-case basis with the given turning movement patterns of a corridor.

See the Abu Dhabi *Access Management Policy and Procedures Manual* (21) for additional guidance.

11.5 Operational Analysis

11.5.1 Introduction

This section presents methods for analysing the operation of an existing or planned roundabout. The methods allow a transportation analyst to assess the operational performance of a facility, given information about the usage of the facility and its geometric design elements. An operational analysis produces two kinds of estimates: (1) the capacity of a facility (i.e. the ability of the facility to accommodate various streams of users) and (2) the level of performance, often using one or more measures of effectiveness.

The *Highway Capacity Manual 2010* (HCM) (17) defines the capacity of a facility as “the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions.” While capacity is a specific measure that can be defined and estimated, level of service (LOS) is a qualitative measure that “characterises operational conditions within a traffic stream and their perception by motorists and passengers.” To quantify LOS, the *HCM* defines specific measures of effectiveness for each highway facility type.

11.5.2 Principles

The operational performance of roundabouts is relatively simple, although the techniques used to model performance can be quite complex. A few features are common to the modelling techniques employed by all analysis tools:

- Drivers must yield the right-of-way to circulating vehicles and accept gaps in the circulating traffic stream. Therefore, the operational performance of a roundabout is directly influenced by traffic patterns and gap acceptance characteristics.

- As with other types of intersections, the operational performance of a roundabout is directly influenced by its geometry. The extent to which this influence is affected in aggregate (e.g. number of lanes) or by design details (e.g. diameter) is discussed in the *HCM* (17).

The following sections discuss these principles in more detail.

11.5.2.1 Effect of Traffic Flow and Driver Behaviour

The capacity of a roundabout entry decreases as the conflicting flow increases. In general, the primary conflicting flow is the circulating flow that passes directly in front of the subject entry. When the conflicting flow approaches zero, the maximum entry flow is given by 3,600 seconds per hour divided by the follow-up headway, which is analogous to the saturation flow rate for a movement receiving a green indication at a signalised intersection. This defines the intercept of the capacity model.

A variety of real-world conditions occur that can affect the accuracy of a given modelling technique. The analyst is cautioned to consider these effects and determine whether they are significant for the type of analysis being performed. For example, the level of accuracy needed for a rough planning-level sizing of a roundabout is considerably less than that needed to determine the likelihood of queue spillback between intersections. Some of these conditions include the following:

1. Effect of Exiting Vehicles. While the circulating flow directly conflicts with the entry flow, the exiting flow may also affect a driver's decision on when to enter the roundabout. This phenomenon is similar to the effect of the right-turning stream approaching from the left side of a two-way stop-controlled intersection. Until these drivers complete their exit manoeuvre or right turn, there may be some uncertainty in the mind of the driver at the yield or stop line about the intentions of the exiting or turning vehicle.

2. Changes in Effective Priority. Where both the entering and conflicting flow volumes are high, limited priority (where circulating traffic adjusts its headways to allow entering vehicles to enter), priority reversal (where entering traffic forces circulating traffic to yield), and other behaviours may occur, and a simplified gap-acceptance model may not give reliable results.

3. Capacity Constraint. When an approach operates over capacity during the analysis period, a condition known as capacity constraint may occur. During this condition, the actual circulating flow downstream of the constrained entry will be less than the demand. The reduction in actual circulating flow may therefore increase the capacity of the affected downstream entries.

4. Origin–Destination Patterns. Origin–destination patterns may have an influence on the capacity of a given entry.

11.5.2.2 Effect of Geometry

Geometry plays a significant role in the operational performance of a roundabout in a number of key ways:

1. Speed. It affects the speed of vehicles through the intersection, thus influencing their travel time by virtue of geometry alone (geometric delay).

2. Number of Lanes. It dictates the number of lanes over which entering and circulating vehicles travel.

3. Approach Roadway Widths. The widths of the approach roadway and entry determine the number of vehicle streams that may form side-by-side at the yield line and govern the rate at which vehicles may enter the circulating roadway.

4. Flow. It can affect the degree to which flow in a given lane is facilitated or constrained. For example, the angle at which a vehicle enters affects the speed of that vehicle, with entries that are more perpendicular requiring slower speeds and thus longer headways. Likewise, the geometry of multilane entries may influence the degree to which drivers are comfortable entering next to one another.

5. Perception. It may affect the driver's perception of how to navigate the roundabout and their corresponding lane choice approaching the entry. Improper lane alignment can increase friction between adjacent lanes and thus reduce capacity. Imbalanced lane flows on an entry can increase the delay and queuing on an entry despite the entry operating below its theoretical capacity.

Consequently, the geometric elements of a roundabout, together with the volume of traffic desiring to use a roundabout at a given time, may determine the efficiency with which a roundabout operates. These elements form the core of commonly used models. Recent research has suggested that while aggregate changes in geometry are statistically significant, minor changes in geometry are masked by the large variation in behaviour from driver to driver. As a result, the extent to which geometry is modelled depends on the available data and the modelling technique employed.

11.5.3 Data Collection

Operational analysis of roundabouts requires the collection or projection of peak period turning-movement volumes. For existing conventional intersections, these can be determined using standard techniques. Operational performance of a roundabout can also be measured directly in the field using a variety of techniques.

Note that field measurement of performance measures may require large sample sizes due to the inherent large variability in delay measures.

11.5.4 Analysis Techniques

A variety of methodologies is available to analyse the performance of roundabouts. All are approximations, and the responsibility is with the analyst to use the appropriate tool for conducting the analysis.

The decision on the type of operational analysis method to employ should be based on a number of factors:

- What data is available?
- Can the method satisfy the output requirements?

Table 11-5 presents a summary of common applications of operational analysis tools, along with the outcome typically desired and the types of input data usually available. Note that the outcome desired is distinct from the output of the analysis tool. For example, the lane configuration is commonly determined through an iterative process of assigning lane configurations as inputs to the analysis tool and then assessing the acceptability of the resultant performance measures.

Table 11-5: Selection of Analysis Tool

Application	Typical Outcome Desired	Input Data Available	Potential Analysis Tool
Planning-level sizing	Number of lanes	Traffic volumes	HCM, deterministic software
Preliminary design of roundabouts with up to two lanes	Detailed lane configuration	Traffic volumes, geometry	HCM, deterministic software
Preliminary design of roundabouts with three lanes and/or with short lanes/flared designs	Detailed lane configuration	Traffic volumes, geometry	Deterministic software
Analysis of pedestrian treatments	Vehicular delay, vehicular queuing, pedestrian delay	Vehicular traffic and pedestrian volumes, crosswalk design	HCM, deterministic software
System analysis	Travel time, delays and queues between intersections	Traffic volumes, geometry	Simulation

Source: (4)

11.5.5 Highway Capacity Manual Method

The analytic method presented in the *Highway Capacity Manual 2010* (HCM) (17) allows the assessment of the operational performance of an existing or planned one-lane or two-lane roundabout given traffic-demand levels. The designer should review the discussion in the *HCM* for further guidance.

11.5.5.1 Volume-to-Capacity Ratio

The volume-to-capacity ratio is a comparison of the demand at the roundabout entry to the capacity of the entry and provides a direct assessment of the sufficiency of a given design. For a given lane, the volume-to-capacity ratio, x, is calculated by dividing the lane's calculated capacity into its demand flow rate, as shown in Equation 11.1. Both input values are in vehicles per hour.

$$\mathbf{x} = \frac{\mathbf{v}}{\mathbf{c}}$$

Equation 11.1: Volume to Capacity Ratio

While the *HCM* does not define a standard for volume-to-capacity ratio, international and domestic experience suggests that volume-to-capacity ratios in the range of 0.85 to 0.90 represent an approximate threshold for satisfactory operation. When the degree of saturation exceeds this range, the operation of the roundabout enters a more unstable range in which conditions could deteriorate rapidly, particularly over short periods of time. Queues that carry over from one 15-minute period to the next may form, and delay begins to increase exponentially.

A volume-to-capacity ratio of 0.85 should not be considered an absolute threshold; in fact, acceptable operations may be achieved at higher ratios. Where an operational analysis finds the volume-to-capacity ratio above 0.85, it is encouraged to conduct additional sensitivity analysis to evaluate whether relatively small increments of additional volume have dramatic impacts on delay or queues. The analyst is also encouraged to take a closer look at the assumptions used in the analysis (i.e. the accuracy of forecast volumes). A higher volume-to-capacity ratio during peak periods may be a better solution than the potential physical and environmental impacts of excess capacity that is unused most of the day.

11.5.5.2 Level of Service

For roundabouts, level of service (LOS) has been defined using control delay with criteria given in Table 11-6. Control delay is the delay associated with vehicles slowing in advance of the intersection, time spent stopped at an intersection, time spent moving up in the queue, and time needed for vehicles to accelerate to their desired speed. LOS F is assigned if the volume-to-capacity ratio of a lane exceeds 1.0 regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay.

As with any intersection evaluations, LOS is one of several measures (along with volume-to-capacity ratios, control delay, queue length, and other measures) that should be used in the comparison of roundabouts to other intersection types.

Table 11-6: Level-of-Service Criteria

Control Delay (s/veh)	Level of Service by Volume-to-Capacity Ratio*	
	v/c ≤ 1.0	v/c > 1.0
0 – 10	A	F
> 10 – 15	B	F
> 15 – 25	C	F
> 25 – 35	D	F
> 35 – 50	E	F
> 50	F	F

Source: (17)

11.5.5.3 Geometric Delay

Geometric delay is a component of delay that is present at roundabouts, but is not taken into consideration under typical *HCM* procedures. Geometric delay is the additional time that a single vehicle with no conflicting flows spends slowing down to the negotiation speed, proceeding through the intersection, and accelerating back to normal operating speed. Geometric delay may be an important consideration in network planning (possibly affecting route travel times and choices) or when comparing operations of alternative intersection types. While geometric delay is often negligible for through movements at a signalised or stop-controlled intersection, it can be more significant for turning movements at those intersections and all movements through a roundabout. Calculation of geometric delay requires knowledge of the roundabout geometry as it affects vehicle speeds during entry, negotiation, and exit.

For LOS calculations, geometric delay is not needed, as the *HCM* defines LOS solely based on control delay. However, if deterministic software or simulation tools are used to estimate travel time along a corridor, geometric delay is inherently included in the estimate of travel time. Care is needed when comparing results between models.

11.6 Safety

The use of roundabouts is a proven strategy for improving intersection safety by eliminating or altering conflict types, reducing crash severity, and causing drivers to reduce speeds as they proceed into and through intersections. Decreased vehicular speeds also decrease the speed differentials with other road users. Understanding the sensitivity of safety of the various geometric design elements and traffic exposure will assist the designer in optimizing the safety of all vehicle occupants, pedestrians, and cyclists. In addition, the use of safety models will facilitate the planning and design of roundabouts by evaluating their safety compared to other intersection types and by quantifying the safety implications of design decisions.

Many studies have found that one of the benefits of the installation of a roundabout is the improvement in overall safety performance. Research has found that roundabouts have reduced crash frequencies for a wide range of settings (urban and rural) and previous forms of traffic control (two-way stop and signal). This is especially evident with less frequent injury crashes. The safety benefit is greater for small- and medium-capacity roundabouts than for large or multilane roundabouts.

The reasons for the increased safety level at roundabouts are:

1. <u>Conflict Points</u>. Roundabouts have fewer vehicular conflict points in comparison to conventional intersections. The potential for high-severity conflicts (e.g. right angle and left-turn head-on crashes) is greatly reduced with roundabouts.

2. <u>Speed</u>. Lower speeds generally associated with roundabouts allow drivers more time to react to potential conflicts, which improves the safety performance of roundabouts. Lower vehicular speeds help reduce crash severity, making fatalities and serious injuries uncommon at roundabouts. Because most road users travel at similar speeds through roundabouts (i.e. have low relative speeds), crash severity can be reduced compared to some traditionally controlled intersections.

3. Pedestrians. Pedestrians need only cross one direction of traffic at a time at each approach as they traverse roundabouts (i.e. crossing in two stages), as compared with many traditional intersections. Pedestrian–vehicle conflict points are reduced at roundabouts; from the pedestrian's perspective, conflicting vehicles come from fewer directions. In addition, the speeds of motorists entering and exiting a roundabout are reduced with good design, increasing the time available for motorists to react and reducing potential crash severity. While multilane crossings still present a multiple threat challenge for pedestrians, the overall lower speed environment helps to reduce the likelihood of collisions.

NCHRP Report 672 Roundabouts: An Informational Guide, Second Edition (50) presents data used to develop safety prediction models for both intersection-level and approach-level analyses. The intersection-level models are based on total and injury collisions; the latter includes fatal and definite injury, but excludes possible injury collisions. The approach-level models are based on all severities combined for several collision types: entering/circulating, exiting/circulating, and approaching.

The intersection-level models can be used to evaluate the safety performance of an existing roundabout and to estimate the expected safety changes of a roundabout. The approach-level models provide the designer with tools for evaluating design options or evaluating the safety performance of specific approaches.

With respect to roundabout geometry, the designer should note the following observations:

- Entering/circulating collisions increase with an increased entry width.
- Entering/circulating collisions decrease with an increase in central island diameter.
- Entering/circulating collisions decrease as the angle between legs increases.
- Exiting/circulating collisions increase with an increasing inscribed circle diameter.
- Exiting/circulating collisions increase with an increasing central island diameter.
- Exiting/circulating crashes increase with an increasing circulating width.
- Approach crashes increase with increasing lane width.

11.7 Geometric Design

11.7.1 Introduction

The geometric design of a roundabout requires the balancing of competing design objectives. Roundabouts operate most safely where the geometry forces traffic to enter and circulate at slow speeds. Poor roundabout geometry has been found to negatively impact roundabout operations by affecting driver lane choice and behaviour through the roundabout. Many of the geometric parameters are governed by the manoeuvring requirements of the design vehicle. Consequently, designing a roundabout is a process of determining the optimal balance between safety provisions, operational performance, and accommodation of the design vehicle.

Figure 11-11 shows a roundabout designed to fit the area and environment.

Figure 11-11: Urban Roundabout

While the basic form and features of roundabouts are usually independent of their location, many of the design outcomes depend on the surrounding speed environment, desired capacity, available space, required numbers and arrangements of lanes, selected design vehicle, and other geometric attributes unique to each individual site. In rural environments where approach speeds are high and cyclist and pedestrian use may be minimal, the design objectives are significantly different from roundabouts in urban environments where cyclist and pedestrian safety are a primary concern. Additionally, many of the design techniques are substantially different for single-lane roundabouts than for multilane roundabouts.

Roundabout design is an iterative process where a variety of design objectives must be considered and balanced within site-specific constraints. Maximizing the operational performance and safety for a roundabout requires the design to think through the design rather than rely upon a design template. The following sections provide ranges of typical values for many of the different geometric elements in the design of individual roundabout components. The use of a design technique not explicitly included in this section or a value that falls outside of the ranges does not automatically create a fatal flaw or unsafe condition provided that the design principles can be achieved.

This section is organised such that the design principles common among all roundabout types are presented first. Even at the concept level, designers are encouraged to develop designs that are consistent with the design principles in order to depict realistic impacts and to better define the required geometry. Poor concepts can lead to poor decision-making at the feasibility stage and can make it more difficult to generate large changes to a design at a later stage.

11.7.2 Principles and Objectives

This section describes the principles and objectives common to the design of all categories of roundabouts. Note that some features of multilane roundabout design are significantly different from single-lane roundabout design, and some techniques used in single-lane roundabout design may not directly transfer to multilane design. However, several overarching principles should guide the development of all roundabout designs.

Achieving the following principles should be the goal of any roundabout design:

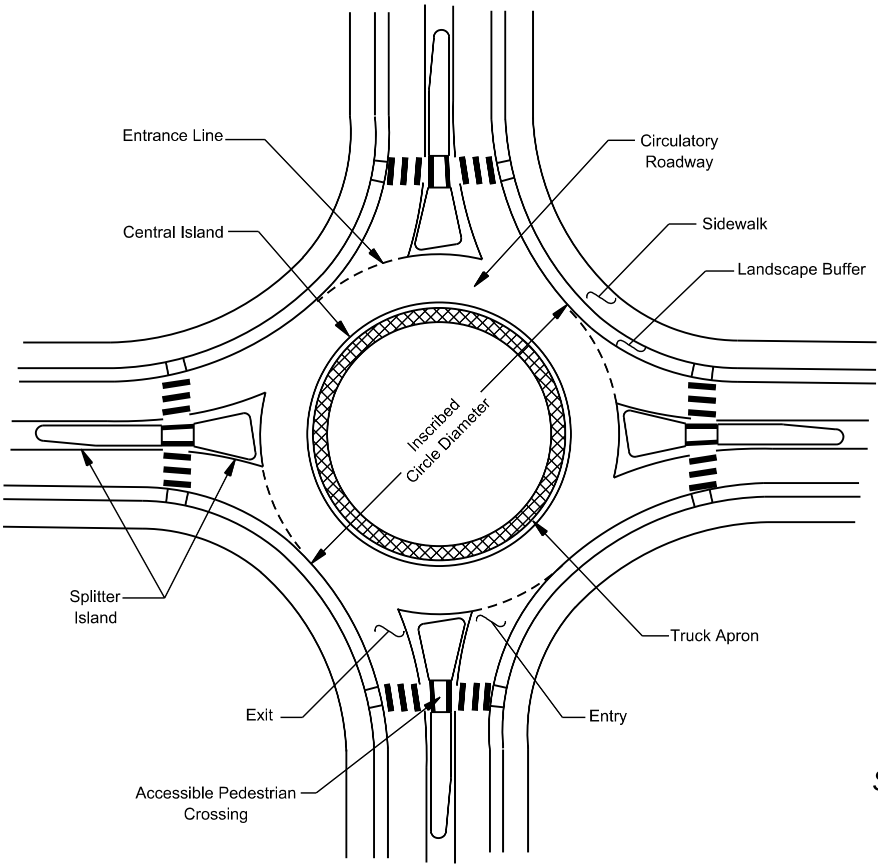
- Provide slow entry speeds and consistent speeds through the roundabout by using deflection.
- Provide the appropriate number of lanes and lane assignment to achieve adequate capacity, lane volume balance, and lane continuity.
- Provide smooth channelization that is intuitive to drivers and results in vehicles naturally using the intended lanes.
- Provide adequate accommodation for the design vehicles.
- Provide a design that meets the needs of pedestrians and cyclists.
- Provide appropriate sight distance and visibility for driver recognition of the intersection and conflicting users.

Each of the principles described above affects the safety and operations of the roundabout. Figure 11-12 illustrates a roundabout that violates several of these principals. When developing a design, the trade-offs of safety, capacity, cost, and so on must be recognised and assessed throughout the design process. Favouring one component of design may negatively affect another. A common example of such a trade-off is accommodating large trucks on the roundabout approach and entry while maintaining slow design speeds. Increasing the entry width or entry radius to better accommodate a large truck may simultaneously increase the speeds that vehicles can enter the roundabout. Therefore, the designer must balance these competing needs and may need to adjust the initial design parameters. To accommodate both the design vehicle and maintain slow speeds, additional design modifications could be required (e.g. offsetting the approach alignment to the left or increasing the inscribed circle diameter of the roundabout).

Figure 11-12: Poorly Designed Roundabout

Figure 11-13 illustrates the basic geometric features and key dimensions of an urban roundabout.

Figure 11-13: Geometric Elements of an Urban Roundabout



Source: (50)

11.7.3 Speed Management

Achieving appropriate vehicular speeds for entering and traveling through the roundabout is a critical design objective as it has profound impacts on safety of all users; it also makes roundabouts easier to use and more comfortable for pedestrians and cyclists. A well-designed roundabout reduces vehicular speeds upon entry and achieves consistency in the relative speeds between conflicting traffic streams by requiring vehicles to negotiate the roundabout along a curved path.

The operating speed of a roundabout is widely recognised as one of its most important attributes in terms of safety performance:

1. Design Speed. Although the frequency of crashes is most directly tied to volume, the severity of crashes is most directly tied to speed. Therefore, careful attention to the design speed of a roundabout is fundamental to attaining good safety performance.

2. Maximum Speed. The maximum entering design speeds, based on a theoretical fastest path, for single-lane roundabouts are 30 km/h to 40 km/h. At multilane roundabouts, maximum entering design speed should be 40 km/h to 50 km/h, based on a theoretical fastest path assuming vehicles ignore all lane lines. These speeds are influenced by a variety of factors, including the geometry of the roundabout and the operating speeds of the approaching roadways. As a result, speed management is often a combination of managing speeds at the roundabout itself and managing speeds on the approaching roadways.

3. Entrance Speed. Reducing the vehicle path radius at the entry (i.e. deflecting the vehicle path) decreases the relative speed between entering and circulating vehicles, which results in lower entering–circulating vehicle crash rates. However, reducing the vehicle path radius at multilane roundabouts can, if not well designed, create poor path alignment (path overlap), greater side friction between adjacent traffic streams, and a higher potential for sideswipe crashes. Therefore, ensure the design encourages drivers to naturally maintaining their lane.

4. Consistent Speed. In addition to achieving an appropriate design speed for the fastest movements, another important objective is to achieve consistent speeds for all movements. Along with overall reductions in speed, speed consistency can help to minimise the crash rate between conflicting streams of vehicles. This principle has two implications:

   - the relative speeds between consecutive geometric elements should be minimised, and

   - the relative speeds between conflicting traffic streams should be minimised.

11.7.4 Lane Arrangements

Section 11.5 provides the methodologies for conducting an operational analysis for a roundabout. An outcome of that analysis is the required number of entry lanes to serve each of the approaches to the roundabout. For multilane roundabouts, ensure the design provides the appropriate number of lanes within the circulatory roadway and on each exit to provide lane continuity.

The allowed movements assigned to each entering lane are key to the overall design. Basic pavement marking layouts should be considered integral to the preliminary design process to ensure that lane continuity is being provided. In some cases, the geometry within the roundabout may be dictated by the number of lanes required or the need to provide spiral transitions. Lane assignments should be clearly identified on all preliminary designs in an effort to retain the lane configuration information through the various design iterations. For guidance on pavement markings through roundabouts, see the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

In some cases, a roundabout designed to accommodate design year traffic volumes, typically projected 20 years from the present, can result in substantially more entering, exiting, and circulating lanes than necessary in the earlier years of operation. To maximise the potential safety during those early years of operation, the designer may wish to consider a phased design solution that initially uses fewer entering and circulating lanes. As an example, the interim design would provide a single-lane entry to serve the near-term traffic volumes with the ability to cost-effectively expand the entries and circulatory roadway to accommodate future traffic volumes. To allow for expansion at a later phase, the ultimate configuration of the roundabout needs to be considered in the initial design. This requires that the ultimate horizontal and vertical design be identified to establish the outer envelope of the roundabout. Lanes are then removed from the ultimate design to provide the necessary capacity for the initial operation. This method helps to ensure that sufficient right-of-way is preserved and to minimise the degree to which the original roundabout must be rebuilt.

11.7.5 Path Alignment

Path alignment at roundabouts draws parallels to conventional intersections and interchanges. At conventional intersections, drivers will tend to avoid driving immediately next to one another as they pass through small radius curves when executing left or right turn movements. The same is true when drivers negotiate a two-lane loop ramp at an interchange. In both cases, the tendency to avoid traveling side-by-side is stronger when one of the vehicles is large (e.g. truck). This overall behaviour also occurs with roundabouts.

As two traffic streams approach the roundabout in adjacent lanes, vehicles will be guided by lane markings up to the entrance line. At the yield point, vehicles will continue along their natural trajectory into the circulatory roadway. The speed and orientation of the vehicle at the entrance line determines what can be described as its natural path. If the natural path of one lane interferes or overlaps with the natural path of the adjacent lane, the roundabout is not as likely to operate as safely or efficiently as possible. The geometry of the exits affects the natural path that vehicles will travel. Overly small exit radii on multilane roundabouts may result in overlapping vehicle paths on the exit.

A good multilane entry design aligns vehicles into the appropriate lane within the circulatory roadway. Likewise, the design of the exits should provide an alignment to allow drivers to intuitively maintain the appropriate lane. These alignment considerations often compete with the fastest path speed objectives. The offside kerbline projection should guide drivers around the central Island

.Vehicle path overlap occurs when the natural path through the roundabout of one traffic stream overlaps the path of another. This can happen to varying degrees, and it can have varying consequences. For example, path overlap can reduce capacity because vehicles will avoid using one or more of the entry lanes. Path overlap can also create safety problems because the potential for sideswipe and single-vehicle crashes is increased. The most common type of path overlap is where vehicles in the left lane on entry are cut off by vehicles in the right lane due to inadequate entry path alignment. However, path overlap can also occur upon the exit from the roundabout where the exit radii are too small or the overall exit geometry does not adequately align the vehicle paths into the appropriate lane.

11.7.6 Size, Position, and Alignment

The design of a roundabout involves optimizing three design decisions to balance the design principles and objectives established in Section 11.7.4. The design decisions are optimizing size, position, and the alignment of the approach legs. There are numerous possible combinations of each element, each with its own advantages and disadvantages. Selection of the optimum combination will often be based upon the constraints of the project site balanced with the ability to adequately control vehicle speeds, accommodate heavy vehicles, and meet the other design objectives.

Producing sketch-level designs of several alternatives aids the designer in identifying these impacts and better evaluating the range of options that are available. It is important to note that where the location of the roundabout has been shifted from the centre of the existing intersection, the approach alignments also require adjustment to achieve entries that are more perpendicular and to achieve speed control.

11.7.6.1 Size

The inscribed circle diameter is the distance across the circle inscribed by the outer curb (or edge) of the circulatory roadway, as illustrated previously in Figure 11-13. It is the sum of the central island diameter and twice the circulatory roadway width. The inscribed circle diameter is determined by a number of design objectives, including accommodation of the design vehicle and providing speed control, and it may require iterative experimentation. Once a sketch-level design concept has been completed, the designer is encouraged to look critically at the design to identify whether the initial assumed diameter produces a desired outcome (e.g. acceptable speeds, adequately serving the design vehicle, appropriate visibility for the central island, minimum size) or whether a larger or smaller diameter would be beneficial. In urban areas, minimum inscribed circle diameters are preferred for land saving considerations and to enhance pedestrian safety.

At single-lane roundabouts, the size of the inscribed circle is largely dependent upon the turning requirements of the design vehicle. The diameter must be large enough to accommodate the design vehicle while maintaining adequate deflection curvature to ensure safe travel speeds for smaller vehicles. However, the circulatory roadway width, entry and exit widths, entry and exit radii, and entry and exit angles play a significant role in accommodating the design vehicle and providing deflection. Careful selection of these geometric elements may allow a smaller inscribed circle diameter to be used in constrained locations. The inscribed circle diameter typically needs to be at least 32 m to accommodate a WB-15 design vehicle. Smaller roundabouts can be used for some local street or collector street intersections, where the design vehicle may be a bus or single-unit truck. For locations that must accommodate a larger WB-20 design vehicle, a larger inscribed circle diameter will be required, typically in the range of 40 m to 46 m. In situations with more than four legs, larger inscribed circle diameters may be appropriate. Truck aprons are typically required to keep the inscribed circle diameter reasonable while accommodating the larger design vehicles.

At multilane roundabouts, the size of the roundabout is usually determined by balancing the need to achieve deflection with providing adequate alignment of the natural vehicle paths. Typically, achieving both of these critical design objectives requires a slightly larger diameter than used for single-lane roundabouts. Generally, the inscribed circle diameter of a multilane roundabout ranges from 46 m to 76 m. For two-lane roundabouts, a common starting point is 49 m to 55 m. Roundabouts with three- or four-lane entries may require larger diameters of 55 m to 100 m to achieve adequate speed control and alignment. Truck aprons are sometimes required to keep the inscribed circle diameter reasonable while accommodating the larger design vehicles.

Mini-roundabouts serve as a special subset of roundabouts and are defined by their small-inscribed circle diameters. With a diameter less than 27 m, the mini-roundabout is smaller than the typical single-lane roundabout. The small diameter is made possible by the use of a fully traversable central island to accommodate large vehicles, as opposed to the typical single-lane roundabout where the diameter must be large enough to accommodate a heavy vehicle within the circulatory roadway (and truck apron if applicable) without it needing to travel over the central island. The small footprint of a mini-roundabout offers flexibility in working within constrained sites. However, it has limitations to where it may be appropriate due to the reduced ability control speeds with the traversable central island. Trade-offs of using the smaller diameter mini-roundabout versus the larger-diameter typical single-lane roundabout should be considered based upon the unique site conditions.

Table 11-7 provides typical ranges of inscribed circle diameters for various site locations.

Table 11-7: Typical Inscribed Circle Diameter Ranges

Roundabout Configuration	Typical Design Vehicle	Common Inscribed Circle Diameter (ICD) Range*
Mini-Roundabout	SU-9	14 m to 27 m
Single-Lane Roundabout	WB-12 WB-15** WB-20**	27 m to 46 m 32 m to 46 m 40 m to 55 m
Multilane Roundabout (2 Lanes)	WB-15** WB-20**	46 m to 67 m 50 m to 67 m
Multilane Roundabout (3 Lanes)	WB-15** WB-20**	61 m to 76 m 67 m to 91 m

* *Assumes 90-degree angles between entries and no more than four legs. List of possible design vehicles is not all-inclusive.*

** *Design vehicle should only be used in rural areas.*

*** *Multilane Roundabouts: From local experience 85m ICD appears to be one of the safest designs when dealing with 3 lane roundabouts especially at higher speeds.*

Source: (50)

11.7.6.2 Position and Alignment

The alignment of the approach legs plays an important role in the design of a roundabout. The alignment affects the amount of deflection (speed control) that is achieved, the ability to accommodate the design vehicle, and the visibility angles to adjacent legs. The optimal alignment is generally governed by the size and position of the roundabout relative to its approaches.

A common starting point in design is to centre the roundabout so that the centreline of each leg passes through the centre of the inscribed circle (radial alignment). This location typically allows the geometry of a single-lane roundabout to be adequately designed so that vehicles will maintain slow speeds through both the entries and the exits. The radial alignment makes the central island more conspicuous to approaching drivers and minimises roadway modification required upstream of the intersection.

Another frequently acceptable alternative is to offset the centreline of the approach to the left (i.e. the centreline passes to the left of the roundabout's centre point). This alignment will typically increase the deflection achieved at the entry to improve speed control. However, designers should recognise the inherent trade off a larger radius (or tangential) exit that may provide less speed control for the downstream pedestrian crossing. Especially in urban environments, it is important to have drivers maintain sufficiently low vehicular speeds at the pedestrian crossing to reduce the risk for pedestrians.

Approach alignments that are offset to the right of the roundabout's centre point typically do not achieve satisfactory results, primarily due to a lack of deflection and lack of speed control that result from this alignment. An offset-right alignment brings the approach in at a more tangential angle and reduces the opportunity to provide sufficient entry curvature. Vehicles will usually be able to enter the roundabout too fast, resulting in more loss-of-control crashes and higher crash rates between entering and circulating vehicles. However, an offset-right alignment alone should not be considered a fatal flaw in a design if speed requirements and other design considerations can be met.

11.7.7 Single-Lane Roundabouts

This section presents specific parameters and guidelines for the design of individual geometric elements at a single-lane roundabout. Many of these same principles apply to the design of multilane roundabouts; however, there are some additional complexities to the design of multilane roundabouts that are described in detail in Section 11.7.10. Individual geometric components are not independent of each other; the interaction between the components of the geometry is more important than the individual pieces. The designer should ensure compatibility between the geometric elements to meet overall safety and capacity objectives.

After an initial inscribed diameter, roundabout location, and approach alignment are identified, the design can be more fully developed to include establishing the entry widths, circulatory roadway width, and initial entry and exit geometry. After the initial designs for the entries and exits on each approach have been laid out, conduct performance checks to evaluate the design versus the principles (including fastest path and design vehicle accommodation) to identify any required design refinements. Based on the performance checks, it may be necessary to perform design iterations to adjust the inscribed circle diameter, approach alignments, roundabout location, and/or entry and exit design to improve the composition of the design.

11.7.7.1 Splitter Islands

Splitter islands (also called separator islands, divisional islands, or median islands) should be provided on all single-lane roundabouts. Their purpose is to provide refuge for pedestrians, assist in controlling speeds, guide traffic into the roundabout, physically separate entering and exiting traffic streams, and deter wrong-way movements. Additionally, splitter islands can be used as a place for mounting signs.

When performing the initial layout of a roundabout's design, a sufficiently sized splitter island envelope should be identified prior to designing the entry and exits of an approach. This will ensure that the design will eventually allow for a raised island that meets the minimum dimensions (e.g. offsets, tapers, length, widths). The control points for the splitter island envelope should be identified prior to proceeding to the design of the entry and exit geometry to ensure that a properly sized splitter island is provided.

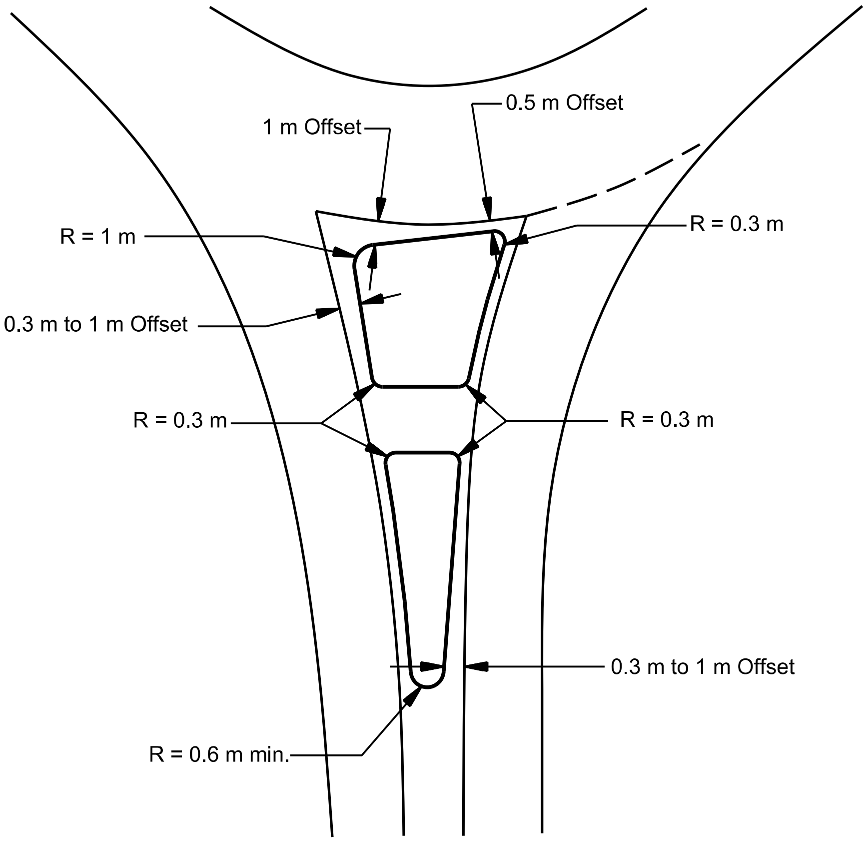
The minimum island width at the pedestrian crossing 2.0 m. The total length of the raised island should generally be at least 15 m, although 30 m is desirable, to provide sufficient protection for pedestrians and to alert approaching drivers to the geometry of the roundabout. On higher speed roadways, splitter island lengths of 45 m or more are often beneficial. Additionally, the splitter island should extend beyond the end of the exit curve to prevent exiting traffic from accidentally crossing into the path of approaching traffic.

There are benefits to providing larger islands. An increase in the splitter island width results in greater separation between the entering and exiting traffic streams of the same leg and increases the time for approaching drivers to distinguish between exiting and circulating vehicles. In this way, larger splitter islands can help reduce confusion for entering motorists. However, increasing the width of the splitter islands generally requires increasing the inscribed circle diameter in order to maintain speed control on the approach. These safety benefits may be offset by higher construction cost and greater land impacts.

See Chapter 10 “Intersections” for island design guidelines for the splitter island. This includes using larger nose radii at approach corners to maximise island visibility and offsetting curb lines at the approach ends to create a funnelling effect. The funnelling treatment aids in reducing speeds as vehicles approach the roundabout. Figure 11-14 shows typical minimum splitter island nose radii and offset dimensions from the entry and exit travelled ways.

Alternative splitter island designs have been adopted by some agencies to meet local design preferences. For instance, some agencies use sloped approach noses, unique curb shapes, and specifications for sloping the top surface of the island outward. Local design standards should be followed in locations where guidance that is more specific has been adopted.

Figure 11-14: Typical Minimum Splitter Island Nose Radii and Offsets



Source: (50)

11.7.7.2 Entry Width

Entry width is measured from the point where the entrance line intersects the left edge of travelled way to the right edge of the travelled way, along a line perpendicular to the right curb line. The width of each entry is dictated by the needs of the entering traffic stream, principally the design vehicle. However, this needs to be balanced against other performance objectives including speed management and pedestrian crossing needs.

Typical entry widths for single-lane entrances range from 4.2 m to 5.5 m; these are often flared from upstream approach widths. However, values higher or lower than this range may be appropriate for a site-specific design vehicle and speed requirements for critical vehicle paths. A 4.6-m entry width is a common starting value for a single-lane roundabout. Care should be taken with entry widths greater than 5.5 m or for those that exceed the width of the circulatory roadway, as drivers may mistakenly interpret the wide entry to be two lanes when there is only one receiving circulatory lane.

11.7.7.3 Circulatory Roadway Width

The required width of the circulatory roadway is determined from the number of entering lanes and the turning requirements of the design vehicle. Except opposite a right-turn-only lane, the circulating width should be at least as wide as the maximum entry width and up to 120% of the maximum entry width. For single-lane roundabouts, the circulatory roadway width usually remains constant throughout the roundabout. Typical circulatory roadway widths range from 4.8 m to 6 m for single-lane roundabouts. Avoid making the circulatory roadway width too wide within a single-lane roundabout because drivers may think that two vehicles are allowed to circulate side-by-side.

At single-lane roundabouts, the circulatory roadway width should be comfortable for passenger car vehicles and should be wide enough to accommodate a design vehicle up to a bus. There may be some operational benefit to accommodating a WB-15 within the circulatory roadway at a single-lane urban arterial roundabout to allow somewhat faster circulating speeds. A truck apron will often need to be provided within the central island to accommodate larger design vehicles (including the WB-20 design vehicles), but maintain a relatively narrow circulatory roadway to adequately constrain vehicle speeds. Use a CADD-based computer program to determine the swept path of the design vehicle through each of the turning movements. Usually, the left-turn movement is the critical path for determining circulatory roadway width. A minimum clearance of 0.3 m and preferably 0.6 m should be provided between the outside edge of the vehicle's tire track and the curb line; see Chapter 10 "Intersections."

11.7.8 Multilane Roundabouts

The principles and design process described previously apply to multilane roundabouts, but in a more complex way. Because multiple traffic streams may enter, circulate through, and exit the roundabout side-by-side, the designer should consider how these traffic streams interact with each other. The geometry of the roundabout should provide adequate alignment and establish appropriate lane configurations for vehicles in adjacent entry lanes to be able to negotiate the roundabout geometry without competing for the same space. Otherwise, operational and/or safety deficiencies may occur.

Multilane roundabout design tends to be less forgiving than single-lane roundabout design. Multilane design can have a direct impact on vehicle alignment and lane choice, which can affect both the safety performance and capacity. Capacity, safety, property impacts, and costs are interrelated, and a balance of these components becomes more difficult with multilane roundabout design. Due to this balancing of design elements that is required to meet the design principles, the use or creation of standard designs is discouraged.

The design of pavement markings and signs at a multilane roundabout is critical to achieving predicted capacities and optimal overall operations. Geometry, pavement markings, and signs must be designed together to create a comprehensive system to guide and regulate road users who are traversing roundabouts. The pavement marking plan should be integral to the preliminary design phase of a project. The Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) provides additional detail on the design of pavement markings and signs for multilane roundabouts.

In addition to the fundamental principles outlined in Section 11.7.4, other key considerations for all multilane roundabouts include:

- lane arrangements to allow drivers to select the appropriate lane on entry and navigate through the roundabout without changing lanes,
- alignment of vehicles at the entrance line into the correct lane within the circulatory roadway,
- accommodation of side-by-side vehicles through the roundabout (i.e. a truck or bus traveling adjacent to a passenger car),

- alignment of the legs to prevent exiting–circulating conflicts, and
- accommodation for all travel modes.

11.7.8.1 Lane Numbers and Arrangements

Multilane roundabouts have at least one approach with at least two lanes on the entries or exits. The number of lanes can vary from approach to approach as long as they are appropriately assigned by lane designation signs and markings. Likewise, the number of lanes within the circulatory roadway may vary depending upon the number of entering and exiting lanes. The important principle is that the design requires continuity between the entering, circulating, and exiting lanes so that lane changes are not required to navigate the roundabout. The driver should be able to select the appropriate lane upstream of the entry and stay within that lane through the roundabout to the intended exit without any lane changes.

The number of lanes provided at the roundabout should be the minimum needed for the existing and anticipated demand as determined by the operational analysis. The designer is discouraged from providing additional lanes that are not needed for capacity purposes as these additional lanes can reduce the safety effectiveness at the intersection. If additional lanes are needed for future conditions, consider using a phased design approach that will allow for future expansion.

On multilane roundabouts, it is also desirable to achieve balanced lane usage in order to be able to achieve predicted capacity. There are a number of design variables that can produce lane imbalance (e.g. poorly designed entry or exit alignments, turning movement patterns). There is a need to recognise possible downstream system variables (e.g. major trip generator, interchange ramp, bottleneck at a downstream intersection). All of these variables may influence lane choice at a roundabout.

11.7.8.2 Entry Width

The required entry width for any given design is dependent upon the number of lanes and the selected design vehicle. A typical entry width for a two-lane entry ranges from 7.3 m to 9.1 m for a two-lane entry and from 11.0 m to 13.7 m for a three-lane entry. Typical widths for individual lanes at entry range from 3.7 m to 4.6 m. The entry width should be primarily determined based upon the number of lanes identified in the operational analysis combined with the turning requirements for the design vehicle. Excessive entry width may not produce capacity benefits if the entry width cannot be fully used by traffic.

For locations where additional entry capacity is required, there are generally two options:

- adding a full lane upstream of the roundabout and maintaining parallel lanes through the entry geometry; or
- widening the approach gradually (flaring) through the entry geometry.

11.7.8.3 Circulatory Roadway Widths

The circulatory roadway width is usually governed by the design criteria relating to the types of vehicles that may need to be accommodated adjacent to one another through a multilane roundabout. The provision of pavement markings within the circulatory roadway (see Abu Dhabi MUTCD (22) may require extra space and the use of a truck apron to support lane discipline for circulating cars and trucks. The combination of vehicle types to be accommodated side-by-side is dependent upon the specific site traffic conditions.

If the entering traffic is predominantly passenger cars and single-unit trucks and where semi-trailer traffic is infrequent, it may be appropriate to design the width for two passenger vehicles or a passenger car and a single-unit truck side-by-side. If semi-trailer traffic is relatively frequent (greater than 10%), it may be necessary to provide sufficient width for the simultaneous passage of a semi-trailer in combination with a passenger or single-unit vehicle.

Multilane circulatory roadway lane widths typically range from 4.0 m to 4.9 m. Use of these values results in a total circulating width of 8.0 m to 9.8 m for a two-lane circulatory roadway and 12.0 m to 14.6 m total width for a three-lane circulatory roadway.

At multilane roundabouts, the circulatory roadway width may be variable depending upon the number of lanes and the design vehicle turning requirements. A constant width is not required throughout the entire circulatory roadway, and it is desirable to provide only the minimum width necessary to serve the required lane configurations within that specific portion of the roundabout. A common combination is two entering and exiting lanes along the major roadway, but only single entering and exiting lanes on the minor street. In this example, the portion of circulatory roadway that serves the minor street has been reduced to a single lane to provide consistency in the lane configurations. For the portions of a multilane roundabout where the circulatory roadway is reduced to a single lane, use the guidance for circulatory roadway width contained in Section 11.7.7.3.

In some instances, the circulatory roadway width may actually need to be wider than the corresponding entrance that is feeding that portion of the roundabout. For example, in situations where two consecutive entries require exclusive left turns, a portion of the circulatory roadway will need to contain an extra lane and pavement markings to enable all vehicles to reach their intended exits without being trapped or changing lanes.

11.7.9 Mini-Roundabouts

A mini-roundabout is an intersection design form that can be used in place of stop control or signalization at physically constrained intersections to help improve safety and reduce delays. Mini-roundabouts are typically characterised by a small diameter and traversable islands and are best suited to environments where speeds are already low and environmental constraints would preclude the use of a larger roundabout with a raised central island.

Mini-roundabouts operate in the same manner as larger roundabouts, with yield control on all entries and counter clockwise circulation around a central island. Due to the small footprint, large vehicles are typically required to travel over the fully traversable central island. To help promote safe operations, the design generally aligns passenger cars to naturally follow the circulatory roadway and minimise running over of the central island to the extent practical.

11.7.9.1 General Design Criteria for Mini-Roundabouts

Many of the same principles are used in the design of mini-roundabouts as in full-sized roundabouts. Key considerations include vehicle channelization, design vehicle paths, and intersection visibility. Given that the central island of a mini-roundabout is fully traversable, the overall design should provide channelization that naturally guides drivers to the intended path. Sub-optimum designs may result in drivers turning left in front of the central island (or driving over the top of it), improperly yielding, or traveling at excess speeds through the intersection.

A mini-roundabout is often considered as an alternative to a larger single-lane roundabout due to a desire to minimise impacts outside of the existing intersection footprint. Therefore, the existing intersection curb lines are a typical starting point for establishing the mini-roundabout inscribed circle diameter. Mini-roundabouts should be made as large as practical within the intersection constraints. However, a mini-roundabout inscribed circle diameter should generally not exceed 30 m. Above 30 m, the inscribed circle diameter is typically large enough to accommodate the design vehicles navigating around a raised central island. A raised central island provides physical channelization to control vehicle speeds; and therefore, a single-lane roundabout design is preferred.

The fully traversable central island provides the clearest indication to the user that the intersection is a mini-roundabout. The location and size of a mini-roundabout's central island, and the corresponding width of the circulatory roadway, is dictated primarily by passenger car swept path requirements. The island location should be at the centre of the of the left-turning inner swept paths, which will be near, but not necessarily on, the centre of the inscribed circle. The off tracking of a large design vehicle should be accommodated by the footprint of the central island; meanwhile, passenger cars should be able to navigate through the intersection without being required to travel over the central island. As with single-lane and multilane roundabouts, it is desirable to accommodate buses within the circulatory roadway to avoid jostling passengers by running over a traversable central island. However, for very small-inscribed circle diameters, the bus turning radius is typically too large to navigate around the central island. For mini-roundabouts with larger inscribed circle diameters, it may be possible to accommodate the swept path of a bus vehicle within the circulatory roadway. The potential trade-off to designing for a bus instead of a passenger car is that the design may result in a wider circulatory roadway and smaller central island.

The location of the central island should allow all movements to be accommodated at the intersection with counter clockwise circulation. Designing the central island size and location to provide deflection through the roundabout will encourage proper circulation and reduced speeds through the intersection.

The central island is typically fully traversable and may either be domed or raised with a sloping curb and flattop for larger islands. Asphalt concrete, Portland cement concrete, or other paving material may be used for the central island and should be domed using a 5% to 6% cross slope, with a maximum height of 125 mm. Although fully traversable and relatively small, it is essential that the central island be clear and conspicuous. Islands with a sloping curb should be designed in a similar manner to truck aprons on typical roundabouts.

The central island should be delineated in accordance with the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

11.7.9.2 Splitter Islands

As with larger roundabouts, splitter islands are generally used at mini-roundabouts to align vehicles, encourage deflection and proper circulation, and provide pedestrian refuge. Splitter islands are raised, traversable, or flush depending on the size of the island and whether trucks will need to track over the top of the splitter island to navigate the intersection. In general, raised islands are used where possible, and flush islands are generally discouraged. The following are general guidelines for the types of splitter islands under various site conditions:

1. Raised Island. Consider a raised island if:
    - all design vehicles can navigate the roundabout without tracking over the splitter island area,
    - sufficient space is available to provide an island with a minimum area of 4.6 m^2, and/or
    - pedestrians are present at the intersection with regular frequency.
2. Traversable Island. Consider a traversable island if:
    - some design vehicles must travel over the splitter island area and truck volumes are minor, and
    - sufficient space is available to provide an island with a minimum area of 4.6 m^2.
3. Flush Island. Consider a flush (painted) island if:
    - vehicles are expected to travel over the splitter island area with relative frequency to navigate the intersection,
    - an island with a minimum area of 4.6 m^2 cannot be achieved, and
    - intersection has slow vehicle speeds.

Where entrance lines are located within the inscribed circle, raised splitter islands typically terminate at the edge of the inscribed circle rather than being carried to the entrance line location. This allows sufficient space within the circulatory roadway for U-turn movements to occur. A painted or traversable splitter island should be continued to the entrance line to guide entering motorists around the central island.

11.7.10 Closely Spaced Roundabouts

It is sometimes necessary to consider the operation of two or more roundabouts in close proximity to each other. In these cases, the expected queue length at each roundabout becomes important (i.e. the expect queue at one roundabout should not block operations at nearby roundabouts). Closely spaced roundabouts may also improve safety by calming the traffic on the major road. Drivers may be reluctant to accelerate to the expected speed on the major road if they are also required to slow again for the next close roundabout. This may benefit nearby residents.

Roundabouts may provide benefit for other closely spaced intersections. Short delay and queuing for vehicles at roundabouts can allow for tighter spacing of intersections without providing a significant operational detriment to the other intersection, if adequate capacity is available at both intersections.

11.8 Other Considerations

11.8.1 Traffic Control Devices

The design installation of traffic control devices is an important component in achieving the desired operational and safety features of a roundabout. The Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) governs the design and placement of traffic control devices, including signs and pavement markings.

11.8.2 Landscaping

Landscaping is one of the distinguishing features that give roundabouts an aesthetic advantage over traditional intersections. Figure 11-15 illustrates landscaping within roundabouts. Landscaping in the central island, splitter islands (where appropriate), and along the approaches can benefit both public safety and community enhancement. In addition to landscaping, Abu Dhabi uses the central island of a roundabout as an opportunity to display local art or other gateway features. To determine the type and quantity of landscaping or other material to incorporate into a roundabout design, consider necessary maintenance requirements and sight distance restrictions. The primary objectives and considerations of incorporating landscaping or art into a roundabout design are to:

- make the central island more conspicuous, thereby improving safety;
- improve the aesthetics of the area while complementing surrounding streetscapes as much as possible;
- make decisions regarding placement of fixed objects (e.g. trees, poles, walls, guide rail, statues, large rocks) that are sensitive to the speed environment in which the roundabout is located;
- avoid obscuring the form of the roundabout or the signing to the driver;
- maintain adequate sight distances;
- clearly indicate to drivers that they cannot pass straight through the intersection;
- discourage pedestrian traffic through the central island; and
- help pedestrians who are visually impaired locate sidewalks and crosswalks.

For additional guidance on roundabout landscaping, see the Abu Dhabi *Road Landscaping Manual* (52).

Figure 11-15: Roundabout Landscaping

11.8.3 Illumination

11.8.3.1 Considerations

Lighting of roundabouts serves two main purposes:

1. It provides visibility from a distance for users approaching the roundabout: and
2. It provides visibility of the key conflict areas to improve users' perception of the layout and visibility of other users within the roundabout.

To improve the users' understanding of the roundabout's operations, the illumination should be designed to create a break in the linear path of the approaching roadway and emphasise the circular aspect features.

- The overall illumination of the roundabout should be approximately equal to the sum of the illumination levels of the intersecting roadways. Local illumination standards should also be considered when establishing the illumination at the roundabout to ensure that the lighting is consistent.
- If continuous roadway lighting is not present, transition lighting should be provided for driver adaption and should be extended along each approach to the roundabout.
- Adequate illumination should be provided on the approach nose of the splitter islands, at all conflict areas where traffic is entering the circulating stream, and at all places where the traffic streams separate to exit the roundabout.
- Adequate illumination should be provided for pedestrian crossing and cyclists merging areas.
- Consideration should be given to the impact of the lighting system in various ambient lighting zones and on adjacent properties. In addition, care should be taken to minimise glare and light trespass.

Provide illumination at all roundabouts, including those in rural environments. Where lighting is not provided, the intersection should be well signed and marked (including the possible use of reflective pavement markers) so that it can be correctly perceived by day and night, recognizing that signing and markings alone cannot correct for the limited view of headlights when circulating.

11.8.3.2 Pole locations

Roundabout light can be achieved by installing lighting within the central island or around the perimeter of the roundabout. The preferred location is to place the lighting poles around the perimeter of the roundabout and at locations on the approach side of crosswalks. Perimeter illumination provides the most optimal visibility within the key conflict areas and visibility of circulating vehicles. Central island lighting may require additional approach lighting. Table 11-8 summarizes some of key advantages and disadvantages for each type of illumination design.

Table 11-8: Advantages and Disadvantages of Perimeter and Central Illumination

Illumination Type	Advantages	Disadvantages
Perimeter Illumination	Illumination can be strongest around critical cyclist and pedestrian areas. Continuity of poles and luminaires is maintained for the illumination of the lanes, as well as good visual guidance on the circulatory roadway. Approach signs typically appear in positive contrast and therefore are clearly visible. Maintenance of luminaires is easier due to curb side location.	Illumination is weakest in the central island, which may limit visibility of roundabout from a distance. More poles are required to achieve the same illumination level. Poles may need to be located in critical conflict areas to achieve illumination levels and uniformity.
Central Illumination	Perception of the roundabout is assisted at a distance by illuminating the central island. Fewer poles are required to achieve the same illumination. Poles in central island are clear of critical conflict areas for all but the smallest of roundabouts. Exit guide signs on the periphery appear in positive contrast (front lit) and, therefore, are clearly visible.	Cannot achieve adequate vertical levels without additional approach lighting. Illumination is weakest in critical pedestrian and cyclist areas. Signs on the approach are in negative contrast (back lit). A path is required to the base of the central pole for maintenance. There is a greater risk of glare. The central pole affects central island landscaping. High mast lighting may be inappropriate in urban areas, in particular, in residential areas.

Source: (50)

In addition, consider the position of lighting poles relative to the curbs at a roundabout and is influenced by speed and possible errant vehicles. In particular, give care to the placement of poles along the exit leg of the roundabout to consider potential paths of errant vehicles. It is desirable to provide adequate clearance to possible hazards. See the Abu Dhabi *Roadside Design Guide* (28) for guidance on clear zones.

11.8.4 Roundabout Visibility

Checking visibility for drivers approaching, driving through or exiting roundabouts is very important safety aspect to be considered when designing roundabouts. Engineers shall consider these checks to ensure different visibility envelopes are clear from any obstruction to line of sight. The following visibility checks to be carried out by designers:

• Forward Visibility on approach

• Forward visibility at entry

• Visibility to the left

- Circulatory Visibility
- Pedestrian crossing visibility

For further details, refer to British Standards for Highways DMRB (Design Manual for Roads and Bridges) Geometric Design of Roundabouts.

12 INTERCHANGES

12.1 Overview

An interchange is a system of interconnecting roadways in conjunction with one or more grade separations that provide for the movement of traffic between two or more roadways on different levels. The operational efficiency, capacity, safety, and cost of the roadway facility are largely dependent upon its design. Chapter 12 provides guidance in the design of interchanges including interchange types, selection, layout, operations, spacing, lane drops, freeway ramp terminals, ramps, and ramp/crossroad terminals. Information that is also applicable to interchanges is included in the following chapters:

- Chapter 10 "Intersections" discusses intersection designs, including left-turn lanes, channelizing islands, turning radii, design vehicles, etc.
- Chapter 7 "Rural and Urban Freeways" discusses freeway types, freeway design criteria, grade separations, and access control along the freeway.

12.2 Warrants

The need for an interchange will vary based on site-specific conditions. Consider the following guidelines when determining the need for an interchange:

1. Access Control. The Abu Dhabi *Access Management Policy and Procedures Manual* (21) provides guidance on access control. The designer should note the following:

   a. Full Access Control. On all fully access-controlled facilities, intersecting crossroads must be terminated, rerouted, provided a grade separation, or provided an interchange. The importance of the continuity of the crossroad, the feasibility of an alternative route, traffic volumes, construction costs, environmental impacts, etc., are evaluated in order to determine which option is most practical. Interchanges generally are provided at:

      - all freeway-to-freeway crossings;
      - all major roads, unless determined inappropriate; and
      - other roads based on the anticipated demand for regional access.

   b. Partial Access Control. On facilities with partial access control (expressways), intersections with public roads will be accommodated by an interchange or with an intersection. Grade separations are rarely provided. Interchanges should be constructed or planned at most major crossroads.

   c. No Access Control. On a facility with no access control, the need for an interchange will be determined on a case-by-case basis emphasising cost effectiveness, safety, and operations. A road-user benefit analysis will generally be required to determine the economic feasibility of an interchange. See Item 5. However, this analysis alone is not sufficient justification for the provision of an interchange.

2. Congestion. Consider providing an interchange where the level of service (LOS) at an intersection is unacceptable and the intersection cannot be redesigned to operate at an acceptable LOS.
3. Safety. In special cases, consider the crash reduction benefits of an interchange at an existing intersection that exhibits extremely high-crash frequencies and rates.
4. Site Topography. Where access is necessary, the topography may dictate an interchange or a grade separation rather than an intersection.
5. Road-User Benefits. If an analysis reveals that road-user benefits over the service life of the interchange will exceed costs, then an interchange may be considered. The designer must consider all costs including right-of-way, construction, maintenance, and user costs in the analysis. For additional guidance, the designer may refer to AASHTO *User and Non-User Benefit Analysis for Highways* (53).
6. Access. An interchange may be required in an area where access availability from other sources is not practical, and the freeway is the only facility that serves the area.
7. Traffic Volumes. Although there are no specific traffic volumes that warrant an interchange, consider providing an interchange where the traffic volumes at an intersection are at or near capacity, and where other improvements are not practical.
8. Urban Streets. For urban grade separations, prior approval from DMT, DMT, and the local municipality is required. Ensure the separation does not adversely impact the surrounding urban context.

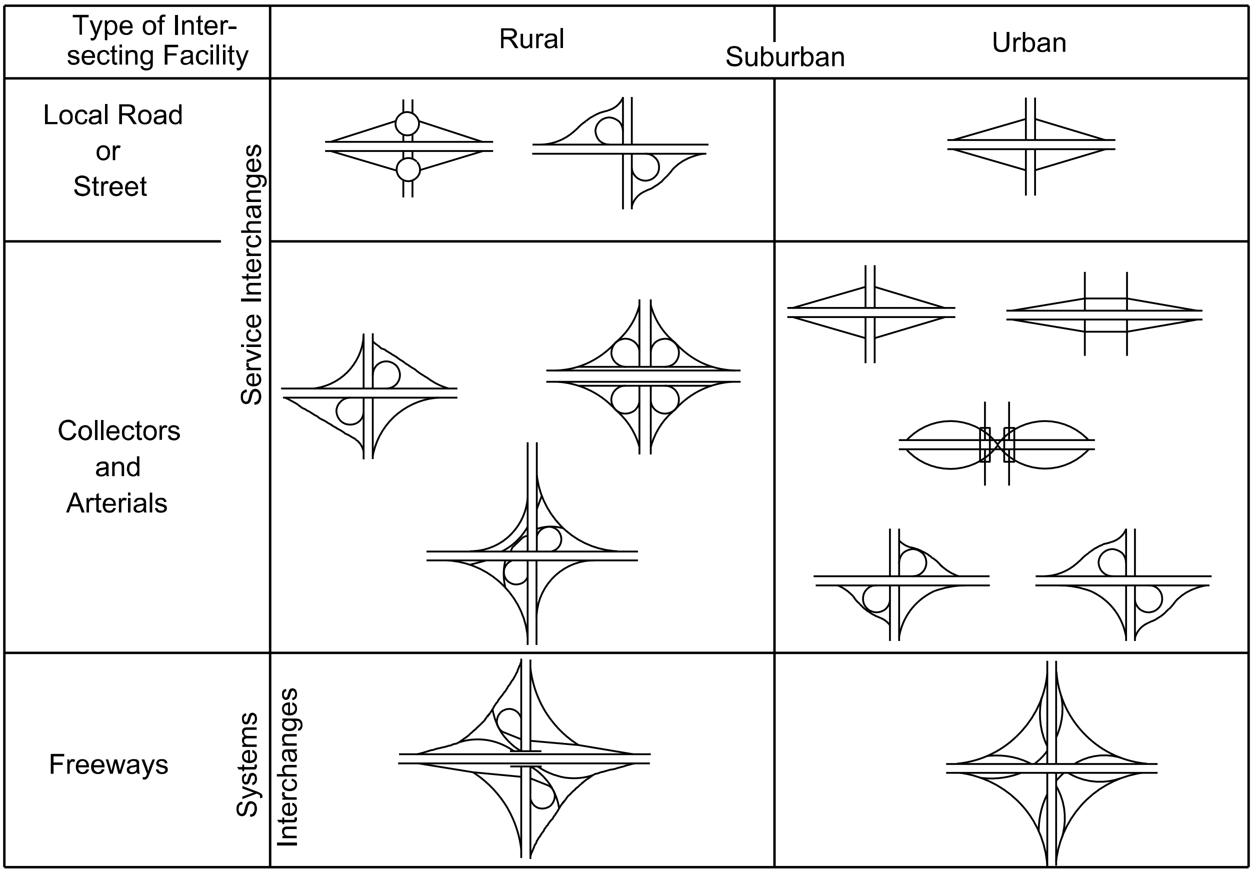
12.3 General Design Considerations

12.3.1 System and Service Interchanges

Interchange configurations are divided into two general categories: system and service interchanges. System interchanges connect a freeway to another freeway or roadways, whereas service interchanges connect a freeway to an arterial, collector, or local roadway. Several configurations of system and service interchanges are shown on Figure 12-1. System interchanges include three- and four- leg directional and trumpet configurations. Service interchange configurations include full and partial clover-leafs, and diamonds.

System interchange designs permit high-speed operation on ramps and typically include major merge and diverge areas on both intersecting roadways. Major merges and diverges typically operate with less turbulence than ramp merge or diverge areas. Service interchanges include lower speed ramp merge and diverge areas, and may include weaving segments on the freeway and crossroad, as well as signalised or unsignalised ramp-crossroad intersections. Table 12-1 summarises the operational features for various interchange configurations. The number of operational features (e.g. diverge/merge areas, weaving areas, ramp/crossroad intersections) are noted for each configuration. For instance, a four-lane directional configuration has four major merge and four major diverge areas, as compared to a full cloverleaf that has two ramp merges, two ramp diverges, and two weave sections on the freeway and crossroad.

Figure 12-1: System and Service Types



Source: (1)

At service interchanges, the operational impacts on the freeway and crossroad need to be considered when assessing various configurations. An optimal freeway design will minimise the number of entry and exit ramps, have high speed ramps to minimise turbulence in merge and diverge areas, and not create short weaving sections. An optimal crossroad design will provide for all turning movements using right-turn movements only, not create short weaving segments, and not require signalised or unsignalised intersections. Left turns are the most difficult to serve in terms of operational efficiency, and high-volume left-turn movements can significantly degrade traffic flow on an interchange crossroad. Combinations of various types of interchanges can be used to minimise capacity reducing features (e.g. weaving areas, left-turns at ramp-crossroad intersections). These combinations can also increase ramp speed in order to minimise turbulence at merge and diverge areas. See the *ITE Freeway and Interchange Geometric Design Handbook* (36) for additional guidance. Figure 12-2, Figure 12-3, and Figure 12-4 illustrate actual service and system interchanges.

Table 12-1: Operational Elements for Various Interchange Types

Interchange	Roadway/ Direction	Diverge		Merge		Weave	Intersections	Left-Turn Movements
		Major	Ramp	Major	Ramp			
4-Leg Directional	Each freeway	4		4				
3-Leg Directional	Through freeway Connecting freeway	2 1		2 1				
Trumpet	Through freeway' Connecting Freeway/Crossroad	2 or 1 or	2 1		2 1			
Full Cloverleaf	Freeway Crossroad		2 2		2 2	2 2		
Partial Cloverleafs								
1-Quadrant, Diamond	Freeway Crossroad		3		2 1		2	3
2-Quadrants, AB	Freeway Crossroad		2		2	1	2	3
2-Quadrants, A	Freeway Crossroad		2		2		2	4
4-Quadrants, A	Freeway Crossroad		2		4		2	2
4-Quadrants, AB	Freeway Crossroad		4		2	1	2	2
4-Quadrants, B	Freeway Crossroad		4		2		2	2
Diamond	Freeway Crossroad		2		2		2	4
Single Point	Freeway Crossroad		2		2		1	4

Source: (36)

Figure 12-2: Service Interchange (Double Roundabout Interchange)

Figure 12-3: Service Interchange (Partial Cloverleaf)

Figure 12-4: System Interchange

12.3.2 Interchange Spacing

Where interchanges are spaced farther apart, freeway operations, level of service, and safety between connecting facilities are improved. Desirably, the spacing between interchanges on the average should not be less than 3 km in urban areas, 6 km in outer urban areas, and 12 km in rural areas. These values allow adequate distances for an entering driver to adjust to the freeway environment, for proper weaving manoeuvres between entrance and exit ramps, and for adequate signing distances. However, considering the effects of existing roads, traffic operations, and social considerations, the spacing between adjacent interchanges may vary considerably. Therefore, the minimum distance between adjacent interchanges should not be less than 1.5 km in urban areas, 3 km in outer urban areas, and 5 km in rural areas. In urban areas, a spacing of less than 1.5 km may be accommodated by using collector-distributor roadways.

12.3.3 Distance Between Successive Freeway/Ramp Junctions

Successive freeway/ramp junctions may be placed relatively close to each other, especially in urban areas. The distance between the terminals should provide for vehicular manoeuvring, signing, and capacity. Figure 12-5 provides guidelines for spacing distances for various freeway/ramp junctions. The criteria in Figure 12-5 should be considered for the initial planning stages of interchange location. The final decision on the spacing between freeway/ramp junctions should satisfy the LOS criteria. This will be determined by conducting a detailed capacity analysis using the *Highway Capacity Manual 2010* (17). Where the distance between the taper of successive entrance and exit terminals is less than 450 m, consider connecting the two terminals with an auxiliary lane, and provide a recovery area beyond the exit terminal.

12.3.4 Basic Number of Lanes

The basic number of lanes is the minimum number of lanes designated and maintained over a significant length of a route based on the overall operational needs of that section. The number of lanes should remain constant over short distances. For example, do not drop a lane at the exit of a diamond interchange and then add it at the downstream entrance simply because the traffic volume decreases between the exit and entrance ramps. Likewise, do not drop a basic lane between closely spaced interchanges simply because the estimated traffic volume does not warrant the higher number of lanes. Lane drops should only occur where there is general lowering of the traffic volumes on the freeway route as a whole.

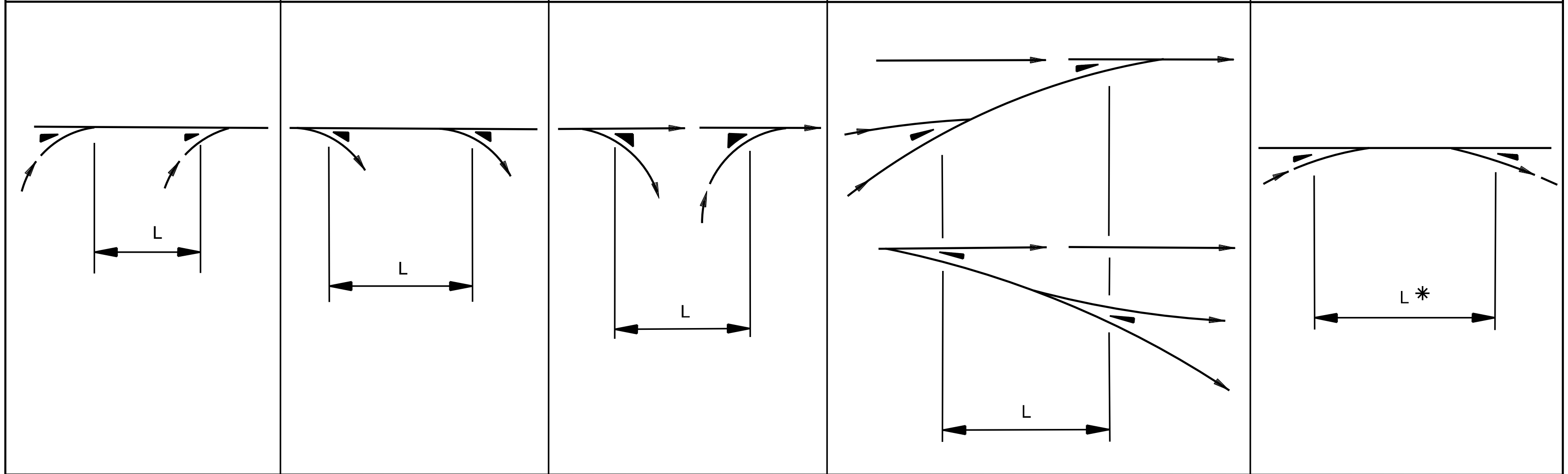
12.3.5 Lane Balance

Lane balance refers to certain principles that apply at freeway exits and entrances:

1. <u>Entrances</u>. At entrances, the number of lanes beyond the merging of the two traffic streams should be not less than the sum of the approaching lanes minus one; see Figure 12-6.

2. <u>Exits</u>. The number of approach lanes on the freeway should equal the sum of the number of mainline lanes beyond the exit plus the number of exiting lanes minus one; see Figure 12-6. An exception to this principle would be at cloverleaf loop ramp exits that follow a loop ramp entrance or at exits between closely spaced interchanges (e.g. interchanges where the distance between the taper end of the entrance terminal and the beginning taper of the exit terminal is less than 450 m and a continuous auxiliary lane is used between the

Figure 12-5: Recommend Minimum Ramp Terminal Spacing



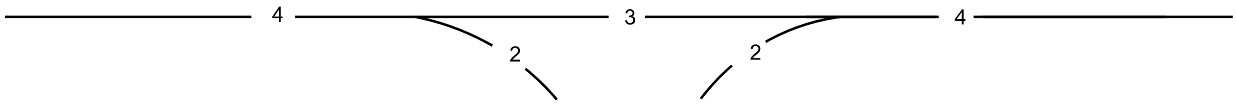
EN-EN		EX-EX		EX-EN		Directional Ramps		EN-EX (Weaving)			
								System to Service Interchange		Service to Service Interchange	
Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	Full Freeway	CDR or FDR	System Interchange	Service Interchange	Full Fwy.	CDR or FDR	Full Fwy.	CDR or FDR
Minimum Lengths (L) Measured Between Successive Ramp Terminals											
300 m		300 m	240 m	150 m	120 m	240 m	180 m	600 m	480 m	480 m	300 m

FDR - Freeway Distributor Road   CDR - Collector-Distributor Road   EN - Entrance   EX - Exit

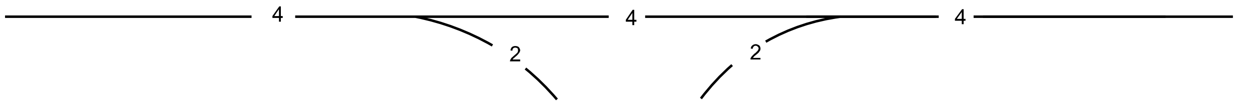
* Not applicable to cloverleaf loop ramps.

Source: (1)

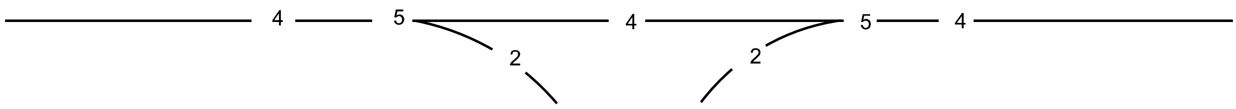
Figure 12-6: Coordination of Lane Balance and Basic Number of Lanes



(a) Lane Balance but no Compliance with Basic Number of Lanes



(b) No Lane Balance but Compliance with Basic Number of Lanes



(c) Compliance with Both Lane Balance and Basic Number of Lanes

Exit: $N_C = N_F + N_E - 1$

Entrance:

Max: $N_C = N_F + N_E$

Min: $N_C = N_F + N_E - 1$

Where:

N_C = Number of Lanes for Combined Traffic

N_F = Number of Lanes on Freeway

N_E = Number of Lanes on Exit or Entrance Ramp

(d) Lane Balance Equations

Source: (1)

terminals). In these cases, the auxiliary lane may be dropped at a single-lane exit with the number of lanes on the approach roadway being equal to the number of through lanes beyond the exit plus the lane on the exit.

3. Travel Lanes. Only reduce the number of travel lanes on the freeway one lane at a time.

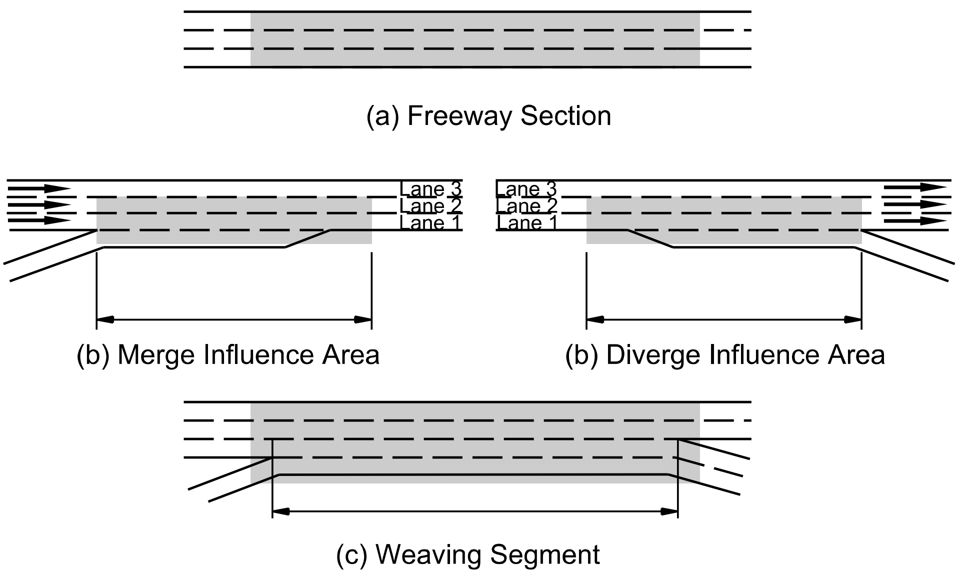
For example, dropping two mainline lanes at a two-lane exit ramp would violate the principle of lane balance. One lane should provide the option of remaining on the freeway. Lane balance would also prohibit immediately merging both lanes of a two-lane entrance ramp into a freeway mainline without the addition of at least one additional lane beyond the entrance ramp. Figure 12-6 illustrates how to coordinate lane balance and the basic number of lanes at an interchange. Figure 12-6 also illustrates how to achieve lane balance at the merging and diverging points of branch connections.

12.3.6 Capacity and Level of Service

The capacity of an interchange will depend upon the operation of its individual elements, which include:

- basic freeway section where interchanges are not present (Section 7.4.3) (Figure 12-7(a));
- freeway ramp terminals, merge and diverge (Section 12.6.3) (Figure 12-7(b));
- weaving areas (Section 12.3.13) (Figure 12-7(c));
- ramp proper (Section 12.5.11); and
- ramp/crossroad intersections (Section **Error! Reference source not found.**).

Figure 12-7: Individual Elements of an Interchange



Source: (17)

The above *Manual* references indicate the locations where basic capacity discussion can be found on each of the listed elements. The *Highway Capacity Manual 2010* (HCM) (17) provides the analytical tools required to analyse the level of service for each element listed above. Other capacity analysis programs and techniques may be used provided they are approved by Abu Dhabi. To be eligible for approval, the output results of other programs and techniques must compare closely with the HCM. Level of service values presented in Chapter 7 “Rural and Urban Freeways” also apply to interchanges. Desirably, the level of service of each interchange element should be equal to the level of service provided on the basic freeway section. Individual elements should not operate at more than one level of service below that of the basic freeway section. In addition, the operation of the ramp/crossroad intersection in urban areas should not impair the operation of the mainline. This will likely involve a consideration of the operational characteristics on the minor road for some distance in either direction from the interchange.

12.3.7 Auxiliary Lanes

As applied to interchange design, auxiliary lanes are most often used to comply with the principle of lane balance, to increase capacity, to accommodate weaving, or to accommodate entering and exiting vehicles. Operational efficiency of the freeway may be improved if a continuous auxiliary lane is provided between entrance and exit terminals where interchanges are closely spaced. An auxiliary lane may be dropped at an exit if properly signed and designed. The following statements apply to the use of an auxiliary lane within or between interchanges:

1. Within Interchange.Figure 12-8 provides the basic schematics of alternative designs for adding and dropping auxiliary lanes within interchanges. The selected design will depend upon traffic volumes for the exiting, entering, and through movements.

2. Between Interchanges. Where interchanges are closely spaced, the designer should provide an auxiliary lane where the distance between the taper end of the entrance terminal and beginning taper of the exit taper is less than 450 m. Figure 12-9 illustrates where an auxiliary lane is used between two closely spaced interchanges.

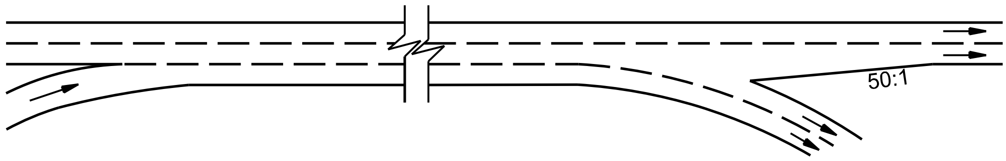
Auxiliary lane drops beyond the interchange may be merged approximately 750 m beyond the influence of the last interchange. Design details for auxiliary lane drops beyond an interchange are provided in Section 12.3.8. Design details for dropping auxiliary lanes at exits or adding them at entrances are discussed in Section 12.6. If the auxiliary lane is dropped at a single lane exit, a recovery area beyond the gore should be provided as shown in Figure 12-8. Where certain sight distance restrictions are unavoidable (e.g. on structures), the recovery area should be extended 150 m to 300 m downstream from the exit. This distance should be increased to 450 m or more with complex designs.

12.3.8 Freeway Lane Drops

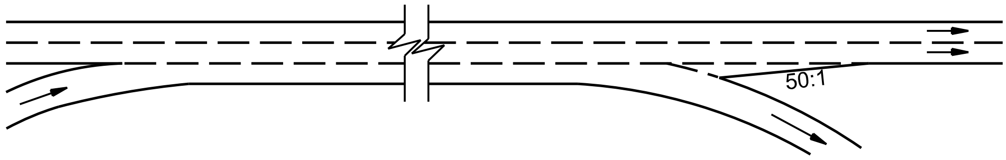
Lane reductions occur when there is a sufficient change in traffic volume in which the basic number of lanes can no longer be justified. Lane drops may occur as the result of:

- the introduction of auxiliary lanes at interchanges,
- in areas where there are multiple interchanges, and/or
- collector-distributor roads necessitating multiple lanes that no longer are required to handle existing or projected traffic volumes.

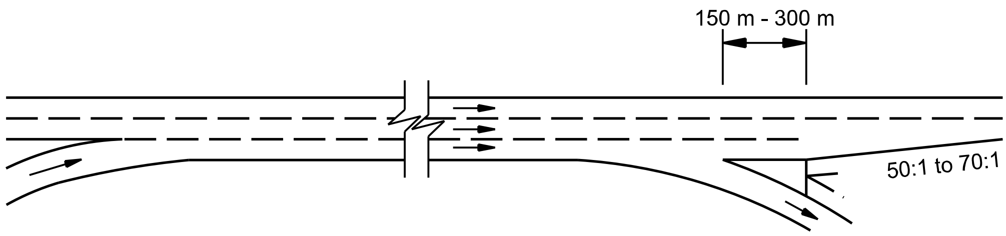
Figure 12-8: Auxiliary Lanes Within an Interchange



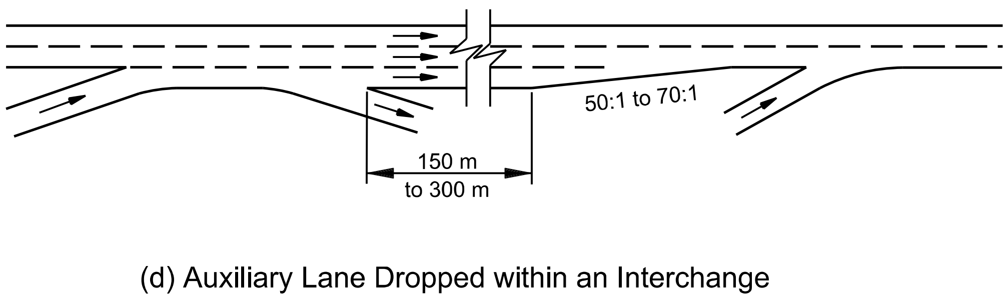
(a) Auxiliary Lane Dropped on Exit Ramp



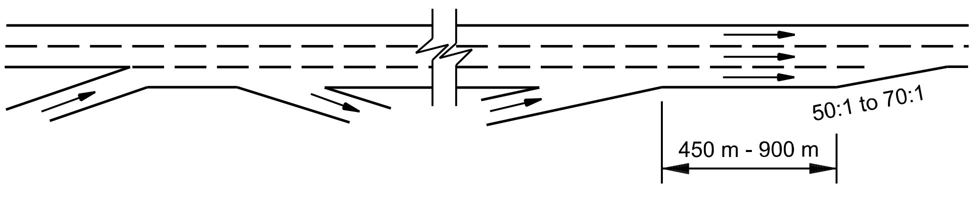
(b) Auxiliary Lane Between Cloverleaf Loops or Closely Spaced Interchanges Dropped on Single Lane (Lane Imbalance for Weaving)



(c) Auxiliary Lane Dropped at Physical Nose (Lane Balanced for Weaving)



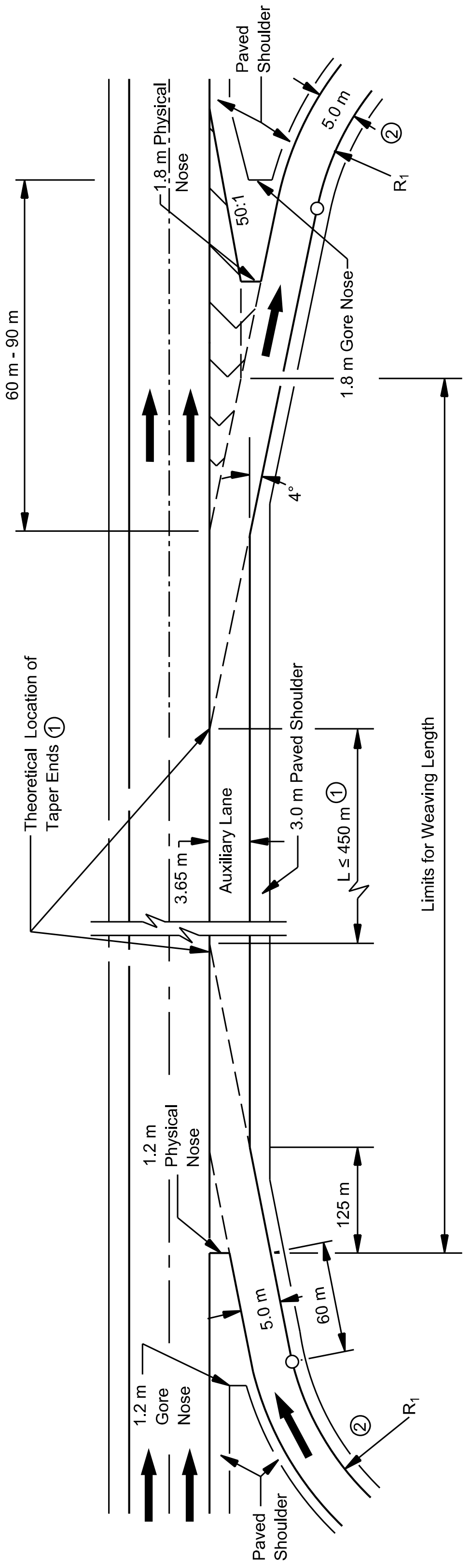
(d) Auxiliary Lane Dropped within an Interchange



Source: (1)
Revised

(e) Auxiliary Lane Dropped Beyond an Interchange

Figure 12-9: Auxiliary Lanes Between Two Interchanges



Notes:

1. *If the distance between the theoretical taper ends is less than or equal to 450 m, provide an auxiliary lane connecting the entrance terminal to the exit terminal even if the auxiliary lane would not be required by a weaving analysis.*
2. *See Sections 12.6.1 and 12.6.2 for one-lane exit and entrance ramps.*

Freeway lane drops should normally occur on the freeway mainline away from any other turbulence (e.g. interchange exits and entrances). Figure 12-10 illustrates the desirable design of a lane drop beyond an interchange. In addition, consider the following criteria when designing a freeway lane drop:

1. Location. The lane drop should occur approximately 600 m to 900 m beyond the previous interchange ramp. This distance allows for adequate signing and driver adjustments from the interchange, but is not so far downstream that drivers become accustomed to the number of lanes, and are surprised by the lane drop. However, if there are a substantial number of trucks, consider providing lengths greater than 900 m. Do not drop a lane on a horizontal curve or where other signing is required (e.g. an DMToming exit).

2. Taper. Lane drops that involve pavement width changes should be transitioned over a length equal to the product of the change in lane width (W) times the design speed (S) times 0.62 (L = W × S × 0.62). Table 12-2 provides lane-drop transition lengths for 3.65-m lanes. Maximum transition taper rate will be 70:1.

Table 12-2: Taper Lengths for 3.65-m Freeway Lane Drop

Design Speed (km/h)	Taper Length (m)
90	200
100	225
120	270
130	295
140	315

3. Sight Distance. Decision sight distance should be available to any point within the entire lane transition. See Section 4.4 for applicable decision sight distance values. These criteria would favour, for example, placing a freeway lane drop within a sag vertical curve or at a location where the freeway lies on an upgrade, but not just beyond a crest.

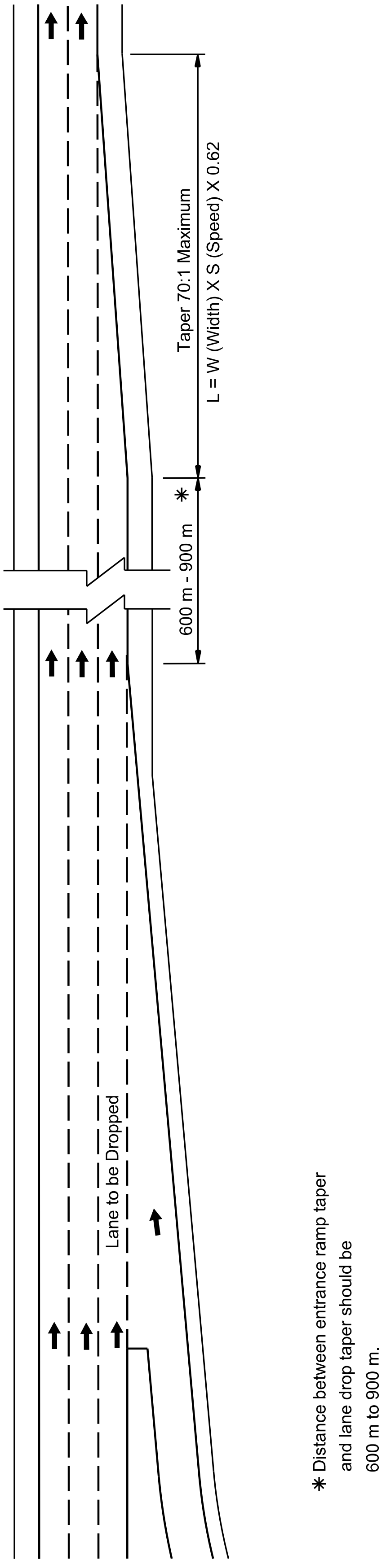
4. Right-Side versus Left-Side Drop. Only provide right-side freeway lane drops because of merging of slower vehicles and normal driver expectations. Do not use a left-side lane drop.

5. Shoulders. Maintain the full-width right shoulder through the lane drop.

12.3.9 Route Continuity

The major route should flow continuously through an interchange. For freeway and expressway routes that change direction, the driver should not be required to change lanes or exit to remain on the major route. Route continuity without a change in the basic number of lanes is consistent with driver expectancy, simplifies signing, and reduces the decision demands on the driver. Interchange configurations should not necessarily favour the heavier traffic movement.

Figure 12-10: Typical Freeway Lane Drop (Right Side)



12.3.10 Uniformity

Interchange configurations should be uniform from one interchange to another. All ramps should exit and enter on the right except under highly unusual conditions. Dissimilar arrangements between interchanges can cause confusion resulting in undesirable lane switches, reduced speeds, hazardous behaviours, etc., especially in urban areas with closely spaced interchanges.

12.3.11 Left-Hand Ramps

Avoid the use of left-hand exit and entrance ramps. They are less efficient operationally than right-hand ramps and may present a serious crash potential. They also introduce an undesirable element of non-uniformity into the design of a freeway system that leads to confusion and, in some cases, hazardous behaviour by drivers. The disadvantages of left-hand ramps greatly outweigh the potential for directional turning movements and the increased flexibility of design.

12.3.12 Signing and Marking

Proper interchange operations depend partially on the compatibility between its geometric design and the traffic control devices at the interchange. The proper application of signs and pavement markings will increase the clarity of paths to be followed, safety, and operational efficiency. The logistics of signing along a roadway segment will also impact the minimum acceptable spacing between adjacent interchanges. The Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) provides guidelines and criteria for the placement of traffic control devices at interchanges.

12.3.13 Weaving Sections

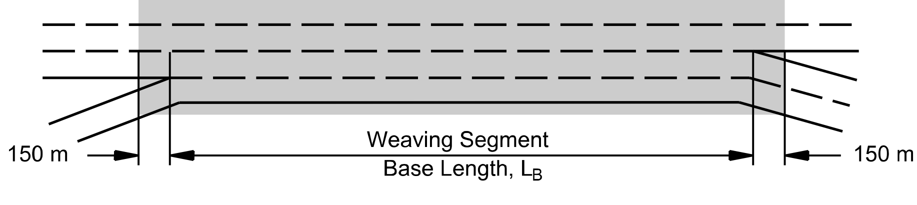
Weaving sections are roadway segments where the pattern of traffic entering and exiting at contiguous points of access results in vehicular paths crossing each other. The turbulent effect of weaving operations can result in reduced operating speeds and levels of service for the through traffic. Desirably, eliminate weaving at an interchange between two major roads by using direct or semi-direct connections or by using collector-distributor roadways.

Consider the following for weaving sections:

1. Weave Length. Weaving sections on freeways other than cloverleafs should be at least 300 m or the length determined using the *Highway Capacity Manual 2010* (17), whichever is greater.

2. Weave Influence Area. The weave influence area is the base length of weave segment (L_B) plus 150 m upstream of entry point to the weaving segment and 150 m downstream of the exit point from the weaving segment. See Figure 12-11. Entry and exit points are defined as points where the appropriate edges of the merging and diverging lanes meet.

Figure 12-11: Weaving Influence Area



Source: (17)

3. Weaving Segment Geometrics. The following geometric characteristics affect the weaving segment's operating characteristics:

    a. Length. Length is the distance between the merge and diverge that forms the weaving segment. The weaving length strongly influences lane-changing intensity. Longer segments allow weaving motorists more time and space to execute lane changes. Lengthening a weaving segment both increases capacity and improves operation.

    b. Width. Width refers to the number of lanes within the weaving segment. Acceleration and deceleration lanes that extend partially into the weaving segment are not included. Although additional lanes provide more space for both weaving and non-weaving vehicles, they may increase lane-changing activity and intensity.

    c. Configuration. Configuration refers to the way entry and exit lanes are aligned. The configuration determines how many lane changes a weaving driver must make to complete the weaving manoeuvre successfully.

4. Maximum Weaving Length. The maximum weaving length (the length where weaving no longer needs to be considered in the capacity analysis) varies based on weaving volumes and the number lanes required to complete the weave. The HCM provides the equation and methodology for determining this distance.

5. Weaving Segment Capacity. Breakdown (capacity) of a weaving segment is expect to occur when:

    - average density of all vehicles in the weaving segment reaches 27 pc/km/lane; or
    - when the total weaving demand flow rate exceeds:
        - 2400 pc/h for two weaving lanes, or
        - 3500 pc/h for three weaving lanes.

6. Level of Service (LOS). The LOS in a weaving segment is related to the density in the segment. Table 12-3 provides the LOS for weaving segments. LOS in weaving segments should be the same as the adjacent mainline; however, at a minimum, it can be one level lower. A higher volume in weaving segments may be accommodated and their adverse impact on through traffic minimised by providing the weaving section on collector-distributor roadways. Section 12.5.2 discusses the use and design of collector-distributor roadways.

Table 12-3: LOS for Weaving Segments

Level of Service	Measures of Performance
	Density (pc/km/ln*)
A	0 – 6
B	6 – 12
C	12 – 17
D	17 – 22
E	22 – 27
F	> 27

**pc/km/ln – passenger cars per kilometre per lane*

Source: (17)

12.3.14 Grading and Landscaping

Consider the grading around an interchange early in the design process. Alignment, fill and cut sections, median widths, lane widths, drainage, structural design, and infield contour grading all affect the aesthetics of the interchange. Properly graded interchanges allow the overpassing structure to blend naturally into the terrain. In addition, ensure that the interchange does not compromise safety, can accommodate blowing sand, can support plantings that prevent erosion, and enhance the appearance of the area. Transitional grading between cut and fill slopes should be long and natural in appearance. The designer must ensure that plantings will not affect the sight distance within the interchange and that larger plantings are a significant distance from the travelled way.

12.3.15 Review Design

12.3.15.1 Review for Ease of Operation

The designer should review the proposed design from the driver's perspective. Examine all possible movements that a motorist might encounter. Several computer modelling programs are available that allow a designer to test drive the design. The designer should review the plans for areas of possible confusion, sufficient weaving and sight distances, proper signing, and ease of operation.

12.3.15.2 Models

Three-dimensional computer and visualization models are helpful in the design of interchanges. Models allow the designer to present the project to others who may not be able to visualize three dimensions from the plans. The use of models also lets the designer to experiment with different concepts.

12.3.16 Geometric Design Criteria

Design all roadways through an interchange with the same criteria as used for the approaches including design speed, sight distance, horizontal and vertical alignment, cross section, and roadside safety elements. Chapter 2 "General Design Criteria and Control," Chapter 3 "Cross Section Elements," Chapter 4 "Sight Distance," Chapter 5 "Horizontal Alignment," and Chapter 6 "Vertical Alignment" present the geometric design criteria that apply to the roadways through interchanges. In addition, consider the following:

1. Design Year. Typically, use a 20-year design period based on the anticipated opening date of the facility.

2. Design Speed. The crossroad design speed will be based on its functional classification and its urban or rural classification; see the geometric design tables in Chapter 7 “Rural and Urban Freeways,” Chapter 8 “Rural Roads,” or Chapter 9 “Urban Streets.”

3. Horizontal Alignment. In general, lay out the alignment of the freeway/expressway and crossroad through the interchange on a tangent. Where this is not practical, consider the following:

    a. Freeway Mainline. Avoid curves to the left if an exit ramp departs straight ahead.

    b. Freeway Ramp Terminals. Curvature and superelevation must be jointly considered to produce appropriate geometry and smooth operations. Lay out the freeway alignment so that only one exit terminal departs from the mainline curving to the right, or design the mainline curve to lie entirely between the exit and entrance terminals.

       A critical exit feature is the shape at the beginning of the terminal, which should be designed to prevent the through driver from inadvertently entering the ramp. This is accomplished by an angular break at the initial taper. The remainder of the terminal is designed as a wrap-inside for right curvature (Figure 12-12(a)) and a wrap-outside for left curvature (Figure 12-12(b)). For right curvature entrance terminals, either a parallel or taper (Figure 12-12(c)) wrap-inside design will work. For a left curvature, Figure 12-12(d) illustrates a parallel wrap-outside design.

    c. Superelevation. Desirably, lay out the horizontal alignment so that superelevation and superelevation transitions will not be required through the freeway ramp terminals or through the ramp/crossroad intersection.

    d. Crossroad. Where a curve is necessary, provide a significantly large horizontal curve so that superelevation is not required on the crossroad.

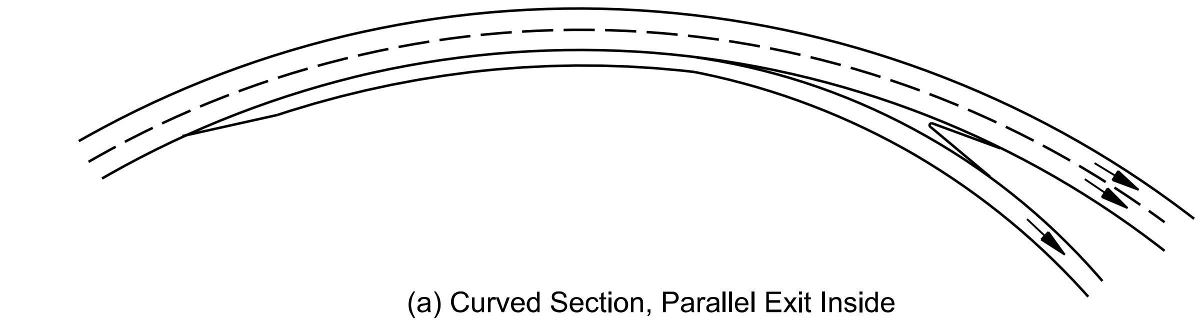
    e. Structures. For a freeway or expressway over a crossroad, place the PC or PT of the horizontal curve 120 m or more from the back of the bridge abutment.

4. Vertical Alignment. Vertical profiles for both roadways through the interchange should be as flat as practical. Where compromises are necessary, use the flatter grade on the major facility. In addition, the designer should consider the following:

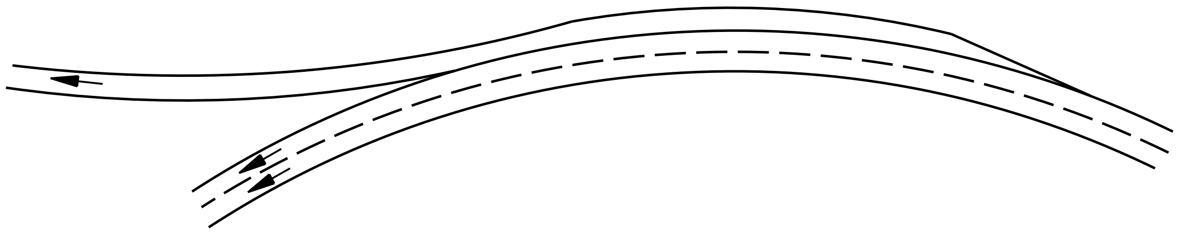
    a. Sight Distance. To improve the sight distance to exit gores, locate exit ramp terminals and major divergences where the mainline is on an upgrade.

    b. Ramps. Avoid creating a hidden ramp roadway in the vertical plane. Also, provide flat approach grades adjacent to the crossroad.

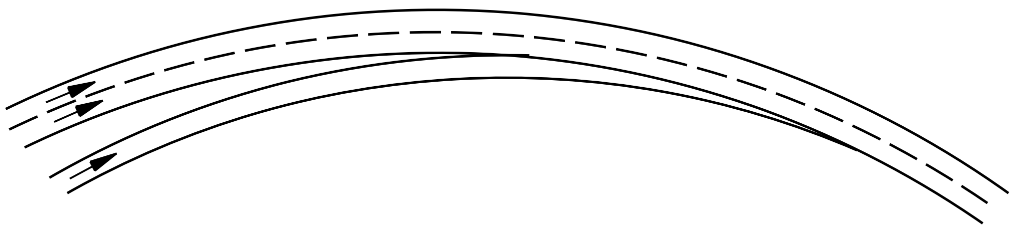
Figure 12-12: Ramp Terminals on Curved Roads



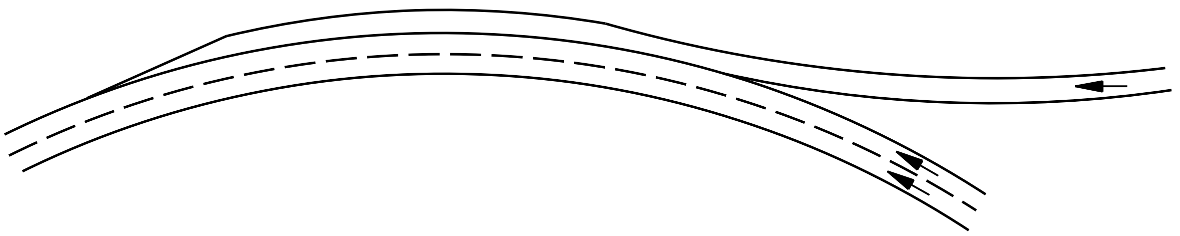
(a) Curved Section, Parallel Exit Inside



(b) Curved Section, Parallel Exit Outside



(c) Curved Section, Tapered Entrance



(d) Curved Section, Parallel Entrance

Source: (36)

c. Exit Ramp Terminals. Where a freeway or expressway is proposed to traverse over the crossroad, locate the exit ramp terminals on the mainline no closer than 300 m from the high point of a crest vertical curve on the mainline. This will ensure that no hidden ramps exist and will provide for safer operations at the exit ramp terminal.

d. Turning Trucks. Large trucks may become unstable when executing a nonstop, left turn from a crossroad on a downgrade. The combination of a downgrade, sharp turning manoeuvres into a ramp, and reverse superelevation may produce instability in large trucks. Therefore, the maximum grade for all crossroads associated with these conditions is desirably 2% through the ramp/crossroad terminal. At a maximum, limit the upgrade and downgrades to 4%.

5. Cross Sections. When designing the crossroad through the interchange, consider the following:

   a. Widths. In general, carry the approach cross section of the major facility through the interchange.

   b. Raised Medians. Raised medians are used throughout the limits of the interchange. This facilitates the construction of separate left-turn lanes and promotes the proper use of the ramp/crossroad intersections. Chapter 10 “Intersections” also provides guidance on the design of channelized left-turn lanes and islands.

   c. Side Slopes. Side slopes on the crossroad through the interchange area should be 1V:4H or flatter. Chapter 3 “Cross Section Elements” discusses roadway side slopes.

6. Sight Distance. Because of the additional demand placed on the driver at an interchange, the designer should consider the following sight distance elements:

   a. Stopping Sight Distance. Provide adequate stopping sight distance on both intersecting roads throughout the interchange and on all ramps. Check both the vertical and horizontal alignment to ensure that the location of piers, abutments, structures, bridge rails, vertical curves, etc., will not restrict the sight distance. Chapter 5 “Horizontal Alignment” discusses the application of horizontal sight distance. Chapter 6 “Vertical Alignment” discusses the application of vertical sight distance.

   b. Decision Sight Distance. Desirably, provide decision sight distance to all decision points (e.g. exit and entrance terminals). Driver expectancy should not be violated; see Section 4.4.

   c. Intersection Sight Distance. Section 4.6 discusses intersection sight distance, which is also applicable at ramp/crossroad intersections (non-merging sites).

7. Ramp/Crossroad Intersections. When designing the ramp/crossroad intersection, consider the following:

   a. Angle of Ramp Intersection. To determine the appropriate angle for the ramp/crossroad intersection, see Section 12.4.2.

b. Access Control. To determine the required length of access control along the crossroad, see the Abu Dhabi *Access Management Policy and Procedures Manual* (21).

c. Left-Turn Lanes. Select the appropriate left-turn lane lengths based on the design speed of the crossroad and/or the required storage lengths; see Chapter 10 “Intersections.”

d. Turning Movements. Check the ramp/crossroad intersection with the applicable design vehicle turning template or use a computer-simulated turning template program. For most ramp/crossroad intersections, design the intersection for the WB-20 design vehicle or, where applicable, the WB-33D design vehicle.

e. Sight Distance. At stopped-controlled intersections, ensure adequate intersection sight distance is available. See Section 4.6.

8. Transit. The designer may want to consider the placement of transit stops within the interchanges so the buses can easily exit and enter the freeway (e.g. separate bus ramps). In addition, ensure pedestrians can easily and safely reach bus stop locations.

9. Trucks. Check truck merging speeds at entrance terminals. This typically is only critical where the:

- mainline profile is on an upgrade of 3% or greater,
- the ramp profile is on a steep upgrade, and/or
- the mainline traffic volume is heavy.

10. Mainline/Crossroad Point of Intersection. Once Items 1 through 9 above have been determined, the designer must decide where the mainline alignment best intersects with the crossroad. The overall size of the interchange, crossroad gradelines, required length of access control along the crossroad, access to property at the ends of access control on the crossroad, and topography are the most influential factors in this determination. Complete this investigation before the detailed design of an interchange is initiated.

12.3.17 Operational/Safety Considerations

Operations and safety are important considerations in interchange design. The following summarises several major considerations:

1. Exit Ramps. For exit ramps, consider the following:

- Provide decision sight distance, where practical, to the freeway exit; see Section 4.4. Desirably, use the pavement surface for the height of object (i.e. 0.0 mm).

- Ramps should depart from the mainline where there will be no vertical curvature to restrict visibility along the ramp. Avoid ramp designs that drop out of sight.

- Avoid locating exit terminals where the mainline curves to the left.

- Proper advance signing of exits is essential to allow necessary lane changes before the exit.

- Provide sufficient distance to allow safe deceleration from the freeway design speed to the design speed of the first governing geometric feature on the ramp, typically a horizontal curve.

2. Entrance Ramps. Provide an acceleration distance of sufficient length to allow a vehicle to attain an appropriate speed for merging. Where entrance ramps enter the mainline on an upgrade, the acceleration distance may need to be lengthened, or an auxiliary lane may be required to allow vehicles to reach a safe speed prior to merging.

3. Driver Expectancy. Ensure that the interchange is designed to conform to the principles of driver expectation. These may include the following:

    - Avoid left-hand exit or entrance terminals. Drivers expect single-lane exit and entrance terminals to be located on the right side of the freeway.

    - Do not locate exit ramps so that it gives the appearance of a continuing mainline tangent as the mainline curves to the left.

    - Do not mix operational patterns between interchanges, lane continuity, or interchange types.

    - Provide lane balance and basic number of lanes on the freeway.

    - Provide sufficient spacing between interchanges to allow proper signing distances to decision points.

4. Fixed Objects. Because of traffic operations at interchanges, fixed objects may be located within interchanges (e.g. signs at exit gores, bridge piers, rails). Avoid locating these objects near decision points, make them breakaway, or shield them with barriers or impact attenuators. Make any concrete footings flush with the ground line. See Abu Dhabi *Roadside Design Guide* (28) for additional guidance.

5. Controlled Ramp Terminals. The designer must ensure that ramp/crossroad intersections have sufficient capacity so that the queuing traffic at the crossroad intersection does not backup onto the freeway. Sufficient access control and intersection sight distance must be maintained along the crossroad to allow the ramp intersection to work properly.

6. Wrong-Way Manoeuvres. Provide channelized medians, islands, and adequate signing to minimise wrong-way possibilities. Avoid designs that may result in poor visibility, confusing ramp arrangements, or inadequate signing.

7. Weaving. Areas of vehicular weaving may create a high demand on driver skills and attentiveness. Where practical, design interchanges without weaving areas by changing the sequence of ramps, increasing the spacing between ramps, or removing the weaving areas from the roadway mainline by using collector-distributor roadways.

8. Pedestrians and Cyclists. Use signing and lane markings to increase awareness of pedestrians and cyclists. Signing, crosswalks, barriers, over and underpasses, bridge sidewalks, and other traffic control devices may be required to manage traffic movements and to control pedestrian and cyclists movements.

12.4 Interchange Types and Layouts

12.4.1 General

The basic interchange types includes the diamond, cloverleaf, partial cloverleaf, trumpet, direct, semi-direct, and roundabout. These interchange types, and variations within each type, permit adaptation to traffic needs, available right-of-way, terrain, and cultural features. The following sections discuss these basic interchange types and design elements for laying out the interchange. Each interchange must be designed to fit the individual site considerations. The final design may be a minor or major modification of one of the basic types or may be a combination of two or more basic types. Sections 12.3, 12.5, 12.6, and 12.7 provide the general design criteria for the individual elements of the interchange.

12.4.2 Diamond Interchange

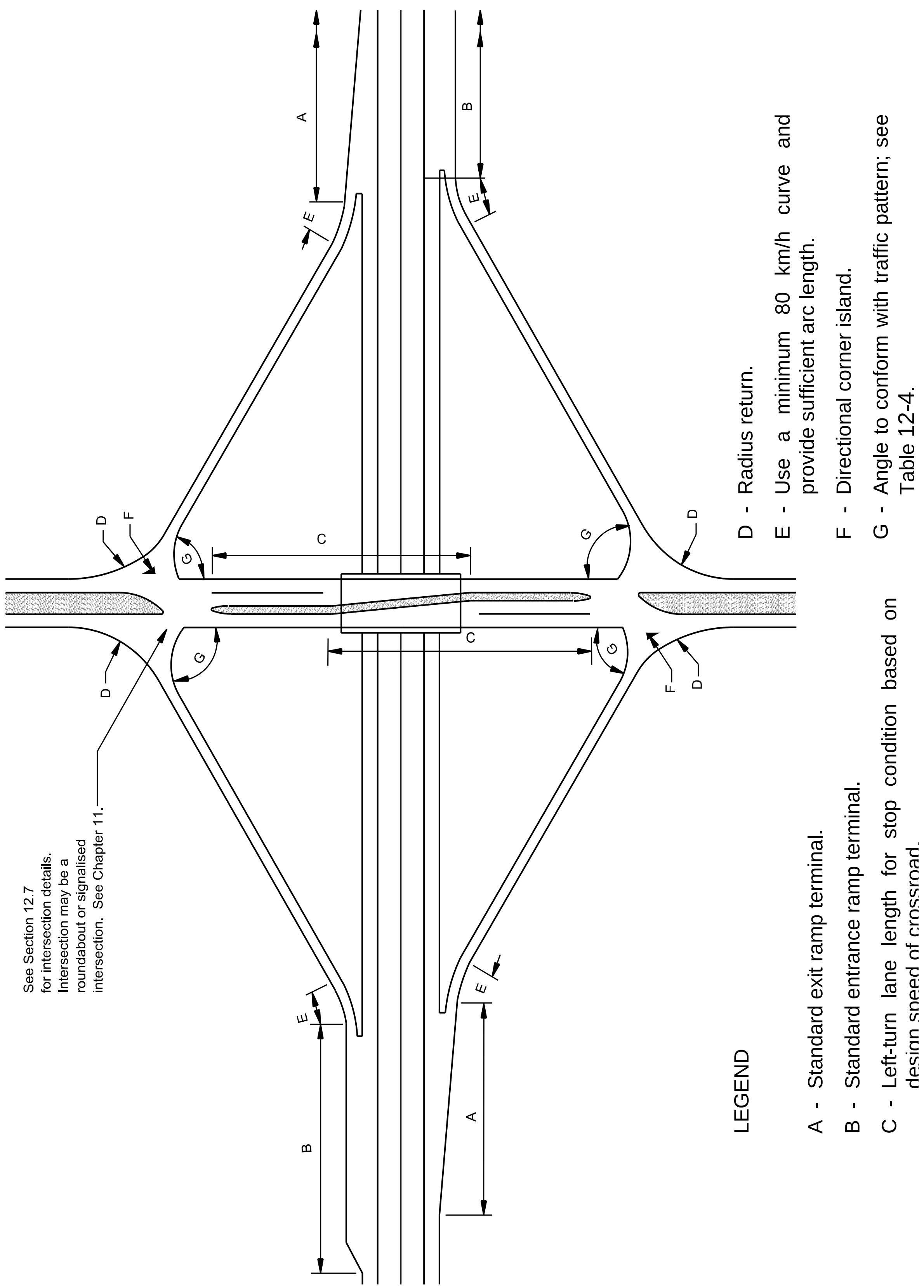
12.4.2.1 General

The conventional diamond is the simplest and most common interchange type. Diamonds include one-way diagonal ramps in each quadrant and two intersections at the crossroad. With proper treatments at the crossroad, the diamond interchange can accommodate a wide variety of circumstances in urban areas where the crossroad operating speeds are 70 km/h or less. The diamond is usually the best interchange choice where the intersecting road is not access controlled. Figure 12-13 illustrates a typical diamond interchange. Some of its advantages and disadvantages include:

<u>Advantages</u>

- All exits from the mainline occur before reaching the crossroad structure and entrances occur after the structure. This conforms to driver expectancy and, therefore, minimises confusion.
- All traffic can enter and exit the mainline at relatively high speeds.
- At the crossroad, adequate sight distance can usually be provided and the operational manoeuvres are consistent with other intersections on the crossroad.
- They require less right-of-way than other interchange types.
- The diamond configuration easily allows modifications to provide greater ramp capacity, if needed in the future.

Figure 12-13: Diamond Interchange



- Their common usage has resulted in a high level of driver familiarity.
- Typically, it is the least expensive of all interchange types.

Disadvantages

- At major ramp/crossroad intersections, a traffic signal is usually required. This will affect the capacity of the intersection.
- Stopped traffic at the ramp/crossroad intersection may cause the ramp traffic to queue onto to the freeway itself.

12.4.2.2 Ramp/Crossroad Intersections

The design criteria presented in Chapter 10 “Intersections” is also applicable to the ramp/crossroad intersection. Table 12-4 provides guidelines for determining the preferred ramp/crossroad intersection angles where the crossroad is approximately perpendicular to the freeway. The ramp angles in Table 12-4 are based on the volume of left-turning vehicles from either the crossroad or ramp. Once the ramp angle is selected, set the left edge of the ramp tangent to the left-turn control radii.

Table 12-4: Ramp/Crossroad Intersection Angles

Left-Turn DHV at Ramp/Crossroad Intersection	Preferred Ramp/Crossroad Intersection Angle
125 or less	60°
125 – 250	75°
250 or more	90°

Note: This table assumes the freeway and crossroad intersect at approximately 90 degrees.

12.4.2.3 Ramp Layout

Although the angle of ramp intersection with the crossroad is important, the complete development of ramp geometry may be influenced by a combination of other factors (e.g. avoiding existing development, angle of intersection of the crossroad with the freeway). Because the angle of the crossroad with the freeway is a major factor in determining the ramp alignment and length, the following design considerations are provided:

1. Freeway and Crossroad — Perpendicular. Where the crossroad is approximately perpendicular to the freeway, consider the following guidelines:

   a. Angle of Intersection. Table 12-4 provides the preferred ramp intersection angles with the crossroad. These angles are based on the number of left-turning vehicles on either the crossroad or the exit ramp.

   b. Number of Curves. With a 60-degree intersection angle, only one curve is required on the ramp; see Figure 12-13. For other angles of intersection, a second curve adjacent to the crossroad will generally be required. For rural ramps with a 75 degree intersection angle with the crossroad, design the ramp curve nearest to the crossroad with a minimum 50 km/h design speed and, desirably, with a 60 km/h design speed. In urban areas, the design speed should be at least 40 km/h unless available right-of-way is highly restricted by existing development.

2. Freeway and Crossroad — Skewed. If the crossroad is skewed, in either direction, strict adherence to the guidelines for perpendicular intersections in Item 1 can result in unacceptable design features (e.g. excessive ramp lengths, short curve lengths, steep grades, indirect alignment). Therefore, design modifications are generally necessary. Under these conditions, give primary consideration to the ramp alignment rather than the intersection angle as determined from Table 12-4. With skewed crossroads, the ramp alignment should be, in order of preference, one of the following:

    - a tangent ramp with a single curve adjacent to the freeway,
    - reverse curves with radii connected by a tangent length greater than the minimum required for superelevation runoff lengths, or
    - reverse curves with radii connected by a tangent equal to the minimum superelevation runoff needs.

    In the design of the preferred alignment, the designer must also control the overall ramp length. Normally, locate the gore nose of a ramp about 375 m from the crossroad structure. If the first preference alignment cannot be developed with a gore within 375 m to 425 m of the structure, investigate the second preference and the third, if necessary, until an acceptable ramp length is achieved for both grade and directness.

12.4.2.4 Double Roundabout Diamond

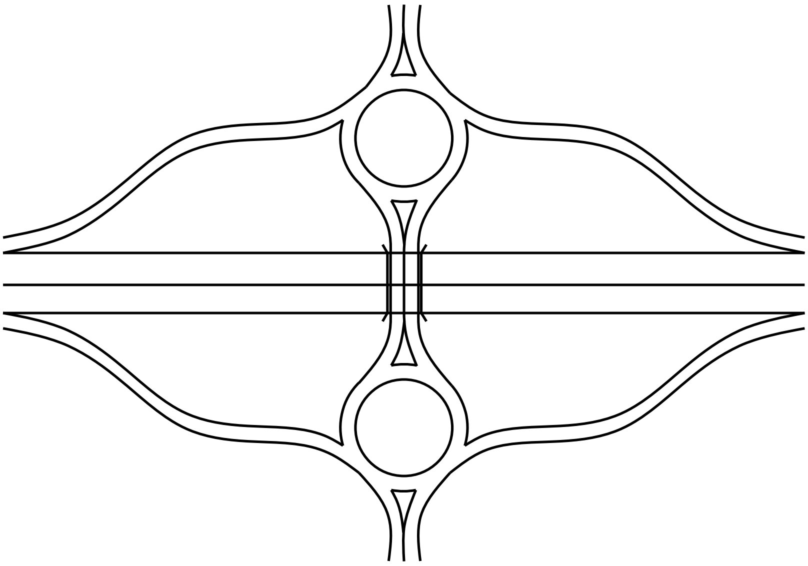
The double roundabout diamond (dumbbell interchange) is a diamond interchange with roundabouts at each crossroad ramp terminal. Figure 12-14 shows two variation of the double roundabout diamond. Free-flow through movements are provided by using two single- or multi-lane roundabouts on the cross street to accommodate left and right turns and all movements on the cross street. The design provides a narrower bridge (no storage turn lanes) and the elimination of signal control at the interchange. Consideration needs to be given to the cross street traffic volumes and freeway ramp volumes when analysing the roundabout operations.

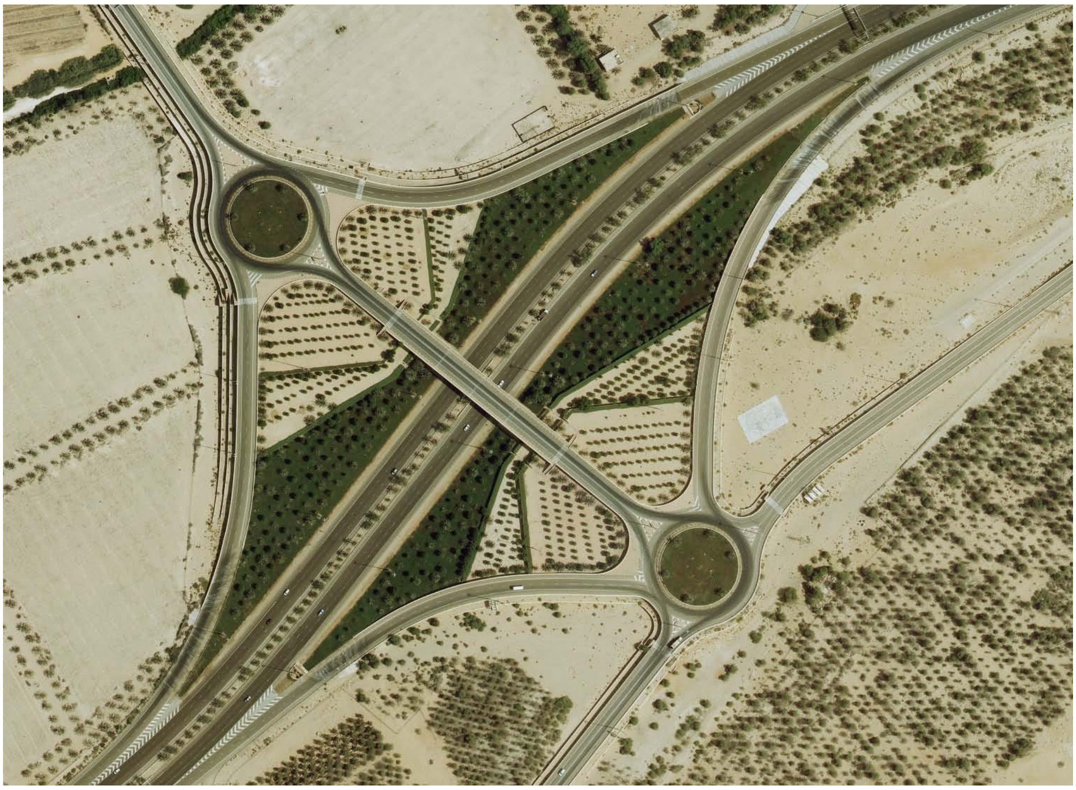
The advantages and disadvantages of the roundabout intersection are similar to those of the full diamond. Some of the advantages and disadvantages of the double roundabout diamond include:

Advantages

- limited right-of-way required,
- easy to sign,
- conventional at-grade intersections,
- single bridge only,
- U-turning possible for main line traffic, and
- no weaving sections on mainline.

Figure 12-14: Double Roundabout Diamond (Dumbbell) Interchanges





<u>Disadvantages</u>

- lower capacity on minor road,
- left-turns interact,
- greater possibility of wrong-way entry to ramp, and
- difficult to expand the intersection in the future.

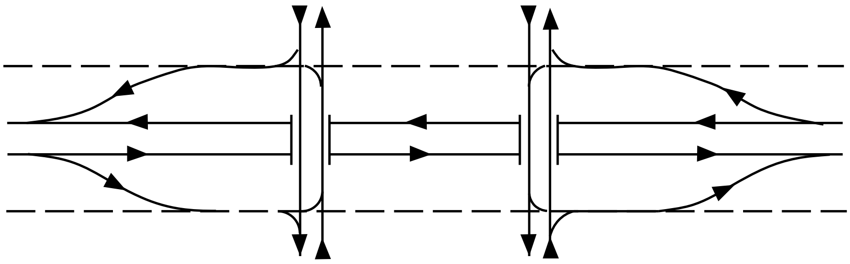
Additionally, the double roundabout diamond eliminates problems arising from the interaction of left turns. However, it has the disadvantage that queues may develop on the off-ramp as other traffic always has priority.

12.4.2.5 Split Diamond

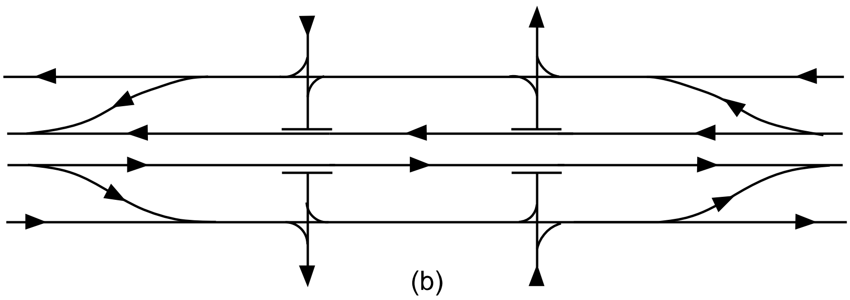
A variation of the conventional diamond is a split diamond interchange. Split diamonds are normally used in urban areas where the designer desires to provide access to two crossing roadway facilities (desirably one-way pairs within 150 m of each other) that are spaced less than 1.5 km apart. Normally, separate interchanges cannot be located within this distance without creating substandard geometric conditions and/or weaving problems without the use of collector-distributor roadways. It is desirable to make the connecting roadways (between the two crossroads) one-way with control of access. Split diamonds have a very desirable feature in that traffic leaving the freeway can return at the same interchange point and continue in the same direction.

Figure 12-15 shows a split diamond in conjunction with a pair of one-way cross streets and one-way frontage roads. The layout and operation of both the crossroad and the at-grade terminals are simplified. Traffic leaving the freeway is afforded easy access to return to the freeway and continue the journey in the same direction.

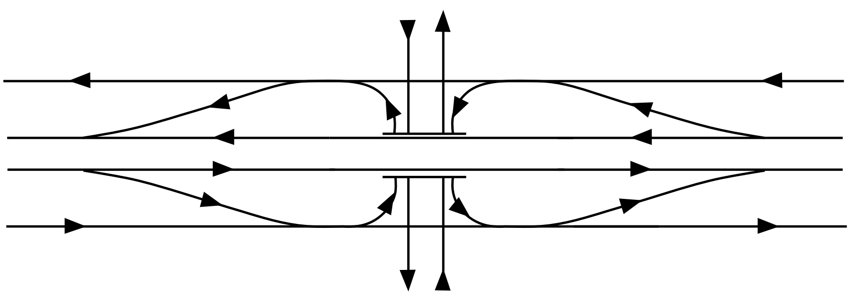
Figure 12-15: Split Diamond Interchanges



(a)



(b)

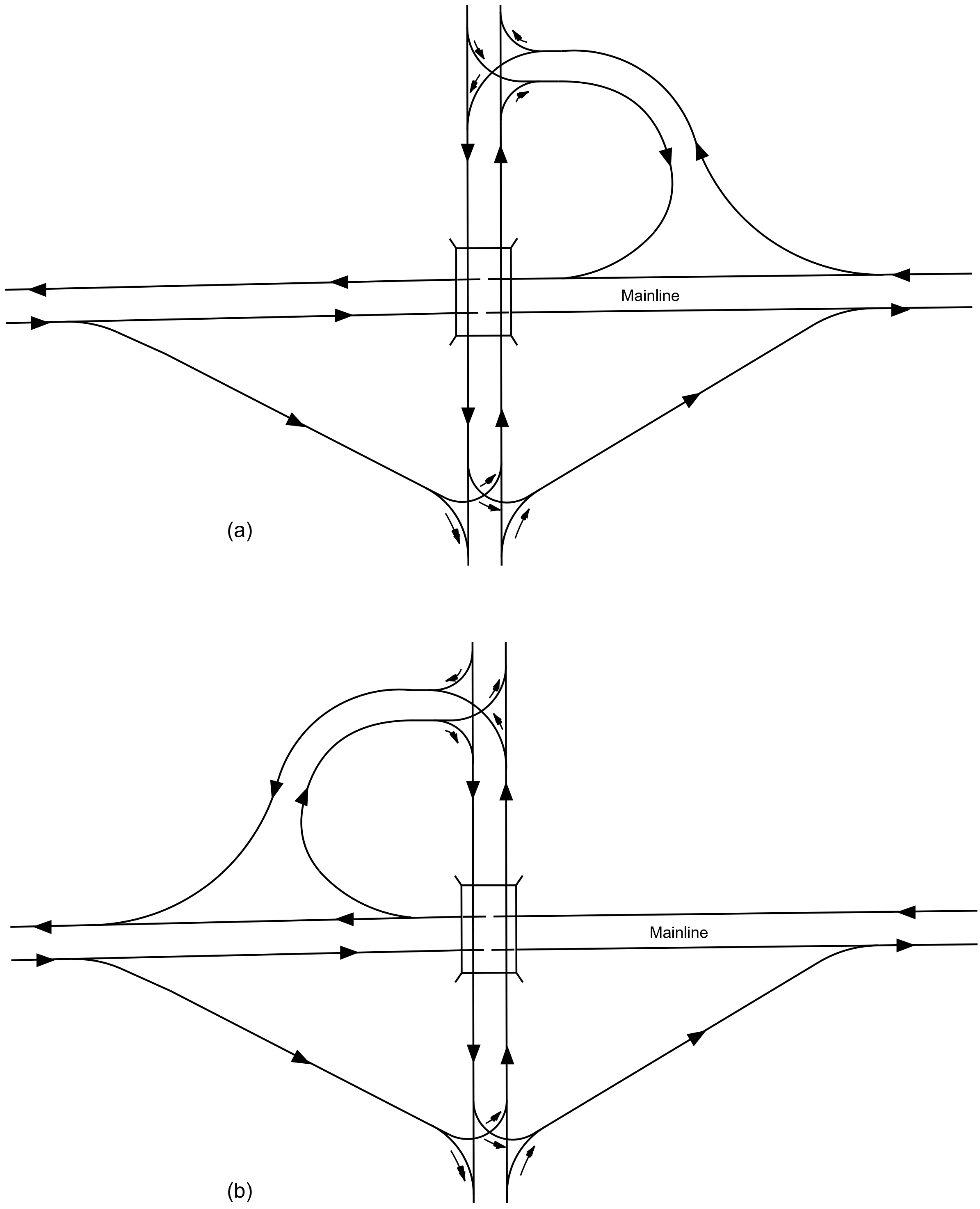


(c)

12.4.3 Modified Diamond

The modified diamond interchange is a combination of the diamond interchange and partial cloverleaf. Figure 12-16 illustrates typical schematics of a modified diamond interchange. This design type is typically used where subdivisions, extensive commercial or industrial development, or other adverse topography and/or soil conditions are located in one of the interchange quadrants, making right-of-way acquisition, design, or construction unusually expensive. Some of the advantages and disadvantages of the modified diamond include:

Figure 12-16: Modified Diamond Interchange



Advantages

- Depending upon site conditions, modified diamonds may offer the opportunity to increase weaving distances.
- It allows access where one of the quadrants presents adverse right-of-way, topography, or environmental constraints.
- It can be used where a full partial cloverleaf is not desirable.

Disadvantages

- Modified diamonds may be more expensive than a conventional diamond interchange due to longer ramp lengths and wider structures.
- The loop results in a longer travel distance for the turning vehicle than for a conventional diamond, and the operating speeds on the loop ramp are generally slower.
- The exit or entrance terminal is located before or after the crossroad structure that may require additional signing to guide the motorist.

12.4.4 Compressed Diamond

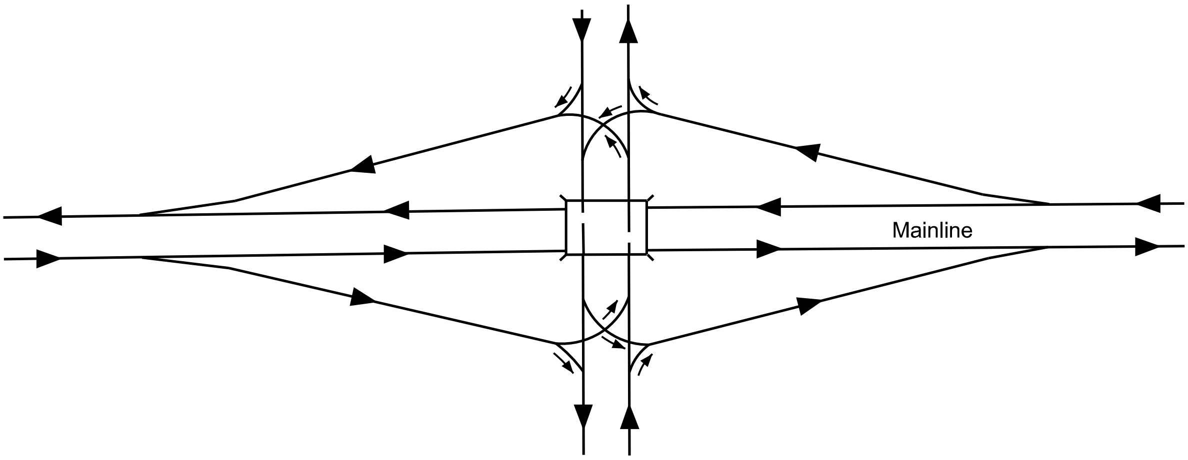
12.4.4.1 General

A compressed diamond, also called a tight diamond interchange, is similar to the conventional diamond except that the ramp termini on the crossroad are located near the structure. Figure 12-17 presents a schematic of a compressed diamond interchange without frontage roads. This design type is generally only used in urban areas where a diamond interchange is appropriate, but right-of-way or other environmental features preclude the use of the conventional diamond. Although operationally a compressed diamond is similar to a single-point diamond discussed in Section 12.4.5, they have significant differences. Some of the advantages and disadvantages of the compressed diamond include:

Advantages

- Less right-of-way is required than that for a conventional diamond.
- The open pavement area at the intersection is significantly less than that for a single point diamond.
- The grade separation structure is significantly smaller than that for a single-point diamond, retaining walls and/or embankments are less expensive, and construction costs are lower.
- The ramp/crossroads intersections operate as two typical intersections, similar to a conventional diamond and, therefore, are less confusing to drivers.
- Slip ramps for one-way frontage roads can be easily incorporated into the design.

Figure 12-17: Compressed Diamond Interchange



Disadvantages

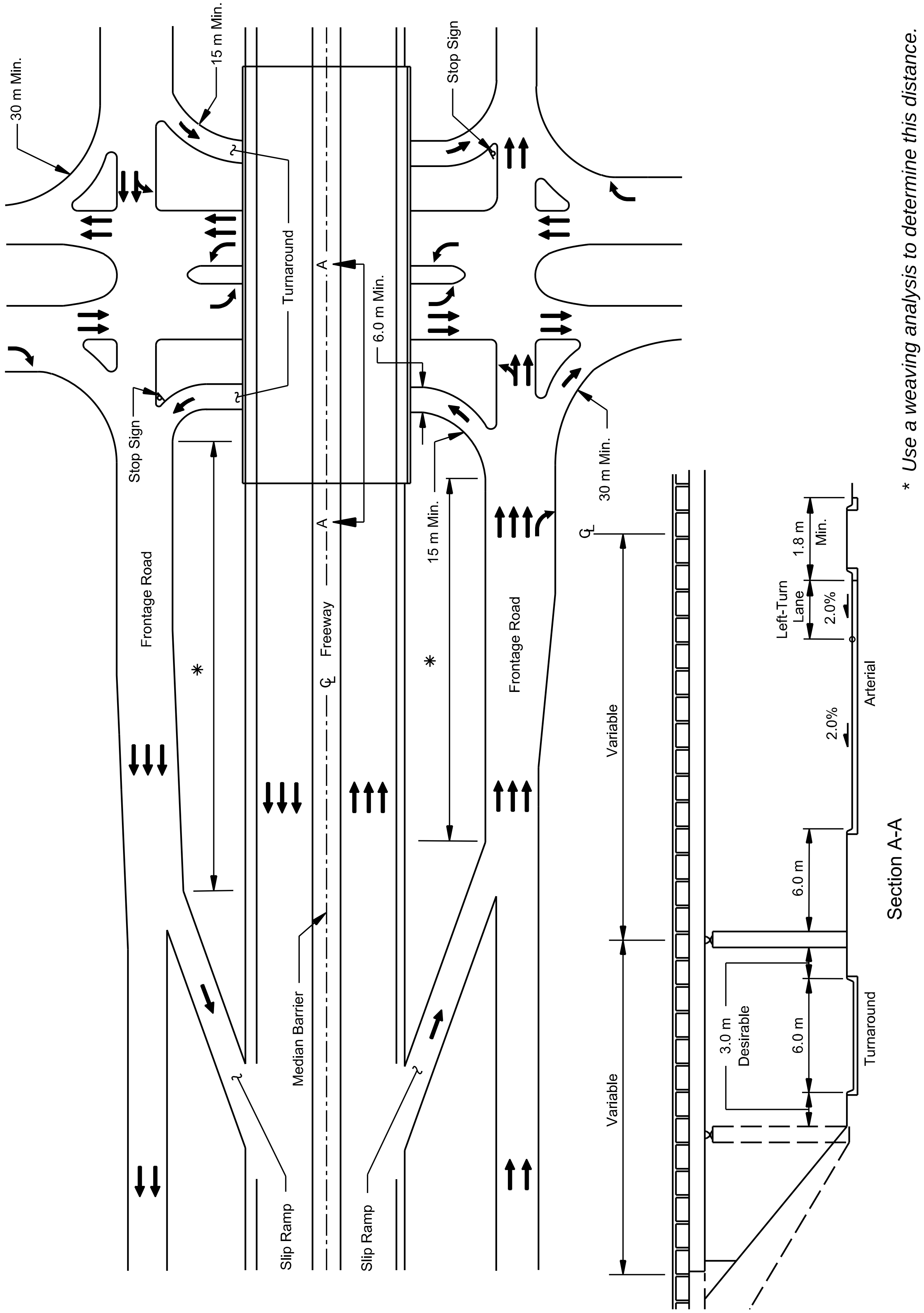
- Left-turn lanes between the ramp termini usually need to be overlapped (i.e. side-by-side opposing left-turn lanes). Consequently, the cross section of the crossroad is generally wider than a conventional diamond.
- Signal timing and interconnection are necessary in order to eliminate left-turn queues from overlapping each other and causing gridlock.
- Due to the close proximity of the two intersections, the compressed diamond typically will need to operate as a six-phase overlap signal system. Consequently, longer clearance times are required.
- Length of access control on the crossroad may be more extensive than that for a conventional diamond.

12.4.4.2 Ramp/Crossroad Intersections

Section 12.7 presents the criteria for ramp/crossroad intersections, which are also applicable to compressed diamonds. However, the minimum length for left-turn lanes is based on the storage length and not on the deceleration distance. See Section 10.7.2 to determine the minimum storage length. If there is insignificant space for storage, the designer will need to consider optimizing the traffic signals.

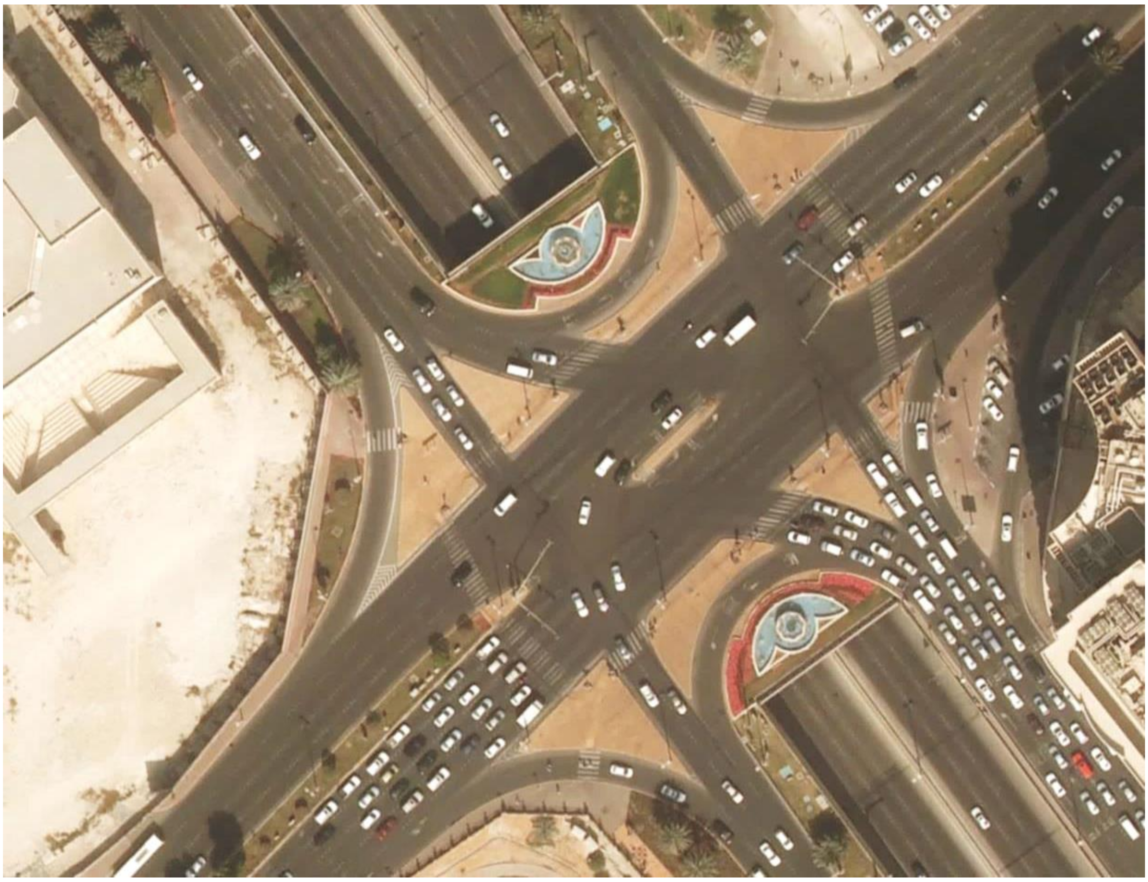
Figure 12-17 illustrates a schematic of a compressed diamond without frontage roads. Where there are one-way frontage roads and where there is significant U-turn traffic to the opposite frontage road, the designer may want to consider using a turnaround design. Figure 12-18, Figure 12-19, and Figure 10-55 illustrate the general layout and cross section for a turnaround design. Depending upon specific site conditions, this arrangement may significantly improve traffic operations at the interchange. The major operational feature of the turnaround is to provide access for traffic on the freeway to the one-way frontage road in the opposite direction without passing through the two intersections on the crossroad.

Figure 12-18: Compressed Diamond with Turnaround



* Use a weaving analysis to determine this distance.

Figure 12-19: Compressed Diamond Interchanges with U-Turns



Some advantages and disadvantages of the turnaround design are as follows:

Advantages

- It preserves and enhances the accessibility to property abutting one-way frontage roads.
- U-turning vehicles do not have to pass through the two intersections on the crossroad.
- The capacity of the crossroad intersections is improved.

Disadvantages

- It is more costly than a typical compressed diamond due to the longer structure.
- It may be confusing to non-repeat drivers because it violates driver expectancy (i.e. driving to the left of the oncoming traffic).
- Longer distances are required between the slip ramp frontage road merge point and the crossroad.

12.4.5 Single-Point Diamond Interchange

12.4.5.1 General

The single-point diamond interchange (SPDI) offers improved traffic-carrying capabilities, safer operations, and reduced right-of-way needs under certain conditions when compared with other interchange configurations. The distinguishing feature of this interchange is the convergence of all through and left-turning movements into a single, large signalised intersection area. Figure 12-20 shows both and over SPDI. Some of the advantages and disadvantages of a SPDI include:

<u>Advantages</u>

- Only requires one intersection instead of two intersections at a typical diamond.
- Allows for better traffic signal progression on the crossroad.
- It can increase interchange capacity and alleviate storage problems from two closely spaced intersections on the crossroad.
- Opposing left turns operate to the left of each other so that their paths do not cross each other.
- Less right-of-way is required than any other interchange type.
- At the intersection of the ramps with the crossroad, the design typically includes flatter curves for turning radii, which allows left turns to be completed at higher speeds.

<u>Disadvantages</u>

- Special pavement markings and a centrally located diamond-shaped island are required to guide the left-turning drivers through the intersection.
- There is a significantly wider pavement area for pedestrians to cross and may create greater delays in traffic when compared to the conventional diamond.
- Because of wide pavement areas, it requires longer signal clearance times.
- It has a higher cost than the conventional or compressed diamond because of the need for a long, single-span structure and the need for retaining walls or reinforced earth walls along the mainline.
- In the case of the mainline over a crossroad, lighting is required under the structure.

12.4.5.2 Design Considerations

The interrelationship of the design elements is extremely important in the design of SPDI. Therefore, make every effort to use the desirable values for all design features of the interchange. See *NCHRP Report 345 Single-Point Urban Interchange Design and Operational Analysis* (54) for complete design details. In addition, consider the following:

Figure 12-20: Single-Point Urban Diamond Interchanges

1. Over Versus Under. One of the first things the designer must address is whether to place the freeway or expressway over or under the crossroad. The overpass SPDI typically includes a conventional, single-span structure 67 m in length with a depth of 2.4 m to 2.7 m. The underpass design (freeway over) typically includes two spans of approximately 20 m in length and a depth of 1.0 m to 1.2 m. The underpass design tends to provide a more open and less restrictive feeling as the driver approaches the intersection area. For both designs, the crossroad profile should be as flat as practical.

2. Sight Distance. Sight distance along the exit ramp to the crossroad intersection is especially critical with the SPDI because the decision point to turn left or right generally will occur sooner at a SPDI than at other diamond type interchanges. The point of initial driver perception of the large triangular intersection island and the point for the left- or right-turn decision should occur at or just beyond the gore nose of the off ramp. At a minimum, provide the stopping sight distance as discussed in Section 4.3 and, desirably, decision sight distance discussed in Section 4.4, wherever practical. The designer must also check the horizontal sight distance to ensure that the structure abutments or parapet walls do not block the sight distance.

3. Intersection Sight Distance. Provide adequate intersection sight distance as discussed in Section 4.6. The designer must check both the vertical and horizontal planes to ensure that adequate intersection sight distance is available. The profile of the crossroad should be flat to allow motorists to see the entire crossroad surface and all ramps in one view.

4. Design Speed. Desirably, the design speed for the turns should be 50 km/h to 60 km/h. In highly restricted ROW areas, the left-turning roadway from the exit ramp onto the crossroad may be designed with a 40 km/h design speed.

5. Horizontal Alignment. One benefit of the SPDI is it provides high-speed, left-turning roadways in comparison to the compressed diamond interchange design. Design the left-turning roadways with 2% superelevation and radii between 60 m and 120 m.

6. Number of Lanes. A capacity analysis is required to determine the number of turn lanes for the overall intersection design. At a minimum, provide sufficient space to allow two through lanes for each direction on the crossroad, one left-turn lane on the crossroad, dual-turn lanes on the exit ramp for left-turning movements, and one right-turn lane from the exit ramp onto the crossroad.

7. Intersection Angle. The intersection angle should be approximately 90 degrees.

8. Median Design. Crossroad medians should be sufficient width to provide for left-turn storage in the median (single or multiple turn lanes).

9. Offset Turning Movements. To allow ease of movements for left-turns, the separation between opposing left-turning vehicles on the crossroad should be at least 3.0 m.

10. Right-Turn Lanes. Ensure the right-turn lanes on the exit ramps are of sufficient length to allow right-turning vehicles to bypass the queue of left-turning vehicles on the ramp.

11. Traffic Control Devices. To eliminate confusion at the SPDI, proper exit ramp guide signing, pavement markings, and lane-use signing must be included to provide the necessary positive guidance through the intersection. Review the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) for the applicable signing and pavement marking criteria.

12. Traffic Signal Placement. When determining signal locations, consider the following:

- Due to possible lane confusion, mount traffic signal heads directly over the travel lanes.
- For overpass SPDI, mount vertical traffic signal heads outside the structure.
- The visibility of the traffic signal heads controlling the exit-ramp left-turning movements is critical. An additional signal for advance notice may be required within the large triangular island.
- A "pull-through" traffic signal on the opposite island may be required where travel distances through the intersection are relatively long.
- Review the Abu Dhabi *Traffic Signals and Electronic Warning and Systems Manual* (45) for the design of all traffic signal installations.

12.4.5.3 Signalisation

Regardless of the diamond configuration, the presence of closely spaced signals, or the one signal in the case of a single-point urban interchange (SPUI), can significantly impact traffic flow on the crossroad and exit ramps. As with any signalised intersection, signal phasing and the allocation of available green time significantly influences the delays and queues that occur for each traffic movement. At an interchange, through and left turn demands on the crossroad and exit ramp approaches vie for available green time. If the timing favours the crossroad, queuing on the exit ramps can extend back onto the freeway, impacting freeway flow and creating an unsafe situation on a high-speed roadway. Conversely, favouring the exit ramps can create long queues on the crossroad, making it difficult for vehicles at access the entry ramps. In order to minimise the impact of queuing, crossroads and exit ramps are often widened to provide dual left-turn lane, right-turn lanes and increase the amount of storage available for through and turning movements.

Signal operation at a conventional diamond interchange is particularly challenging because the two signalised intersections do not operate independently. Due to the relatively short distance between the two intersections, queuing at the downstream signal can impact traffic flow entering the interchange at the upstream signal. In addition, multi-phase operation at both intersections is necessary due to the high left-turn demands that are inherent with the diamond configuration. This phasing requirement can severely limit the traffic progression provided on the crossroad. Three-phased operation provides a more efficient use of time, but can lead to queuing problems between the two intersections, particularly if the separation distance is short. Diamond interchanges with short separation between the two intersections, including the compressed and tight urban designs, typically use a four-phased scheme for this reason. Modified four-phased schemes that include overlap phases are used to improve operational efficiency of a diamond interchange signal.

12.4.6 Full Cloverleaf

12.4.6.1 General

Cloverleaf interchanges are used at four-leg intersections and employ loop ramps to accommodate left-turn movements. Full cloverleaf interchanges are those with loops in all four quadrants; all others are partial cloverleafs and are discussed in Section 12.4.7.

Where two access-controlled roads intersect, a full cloverleaf is the minimum type of interchange design that will suffice. In addition, they also may be used at the intersection of other multilane arterials to accommodate large volumes of traffic.

A major concern with cloverleafs is the weaving problems between the entrance ramp and the following exit ramp. The designer needs to ensure sufficient weaving distances are provided between the entrance and exit terminal. Longer weaving distance will result in larger loops and, thereby, significantly more right-of-way for the interchange. Where this is not practical, the designer should consider removing the weaving from the through lanes by the use of collector-distributor (C-D) roadways; see Section 12.5.2. Safety issues are significantly reduced and weaving volumes are typically increased due to the lower speeds on the C-D roadways. Because of their right-of-way requirements and weaving problems, full cloverleafs are generally only used in rural areas.

Figure 12-21 and Figure 12-22 illustrate typical examples of full cloverleafs with and without C-D roads.

Some of the advantages and disadvantages of full cloverleafs include:

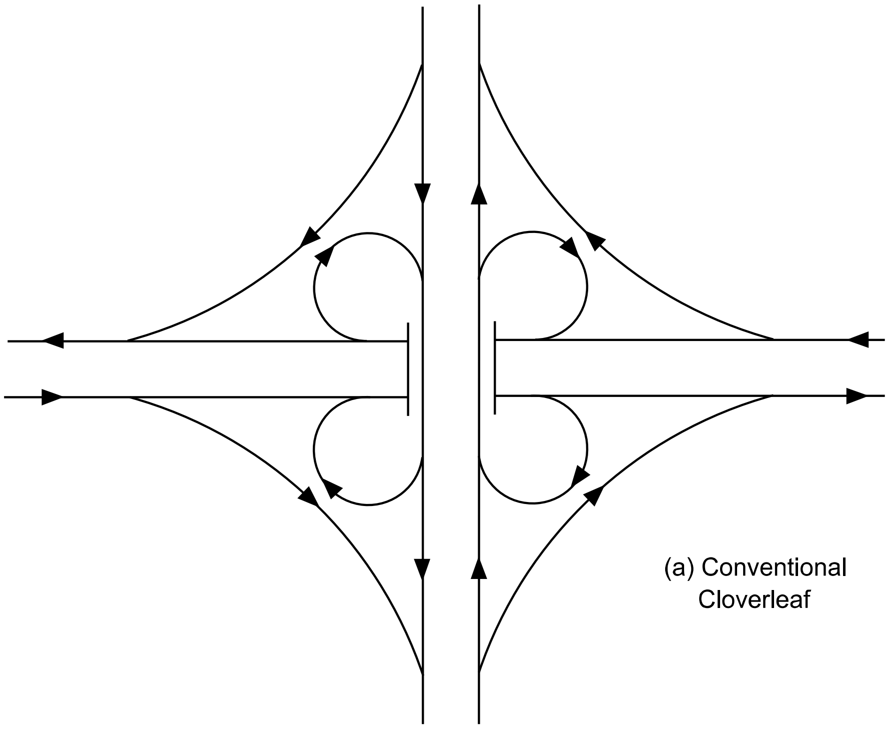
<u>Advantages</u>

- Full cloverleafs eliminate all vehicular stops by using free-flow terminals. They provide continuous free-flow operation on both intersecting roads.
- Full cloverleafs eliminate all at-grade intersections, eliminate left turns across traffic and, therefore, eliminate the need for traffic signals.

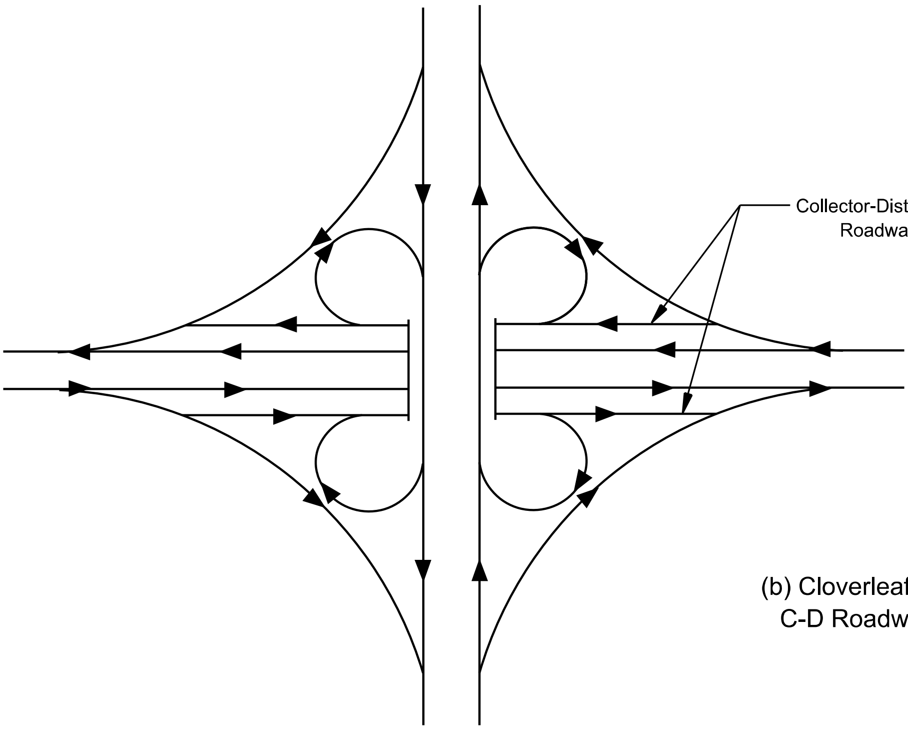
<u>Disadvantages</u>

- Because of the geometric design of loops, full cloverleafs require large amounts of right-of-way.
- They are typically more expensive than diamond interchanges due to considerably more lengths of ramps, wider structures, and the desirability of providing C-D roads.
- The loops in cloverleafs result in a greater travel distance for left-turning vehicles than do diamonds and the speeds on the ramps are generally slower.
- Exit and entrance terminals are located before and after the crossroad structure, which require additional signing to guide motorists.

Figure 12-21: Cloverleaf Interchanges

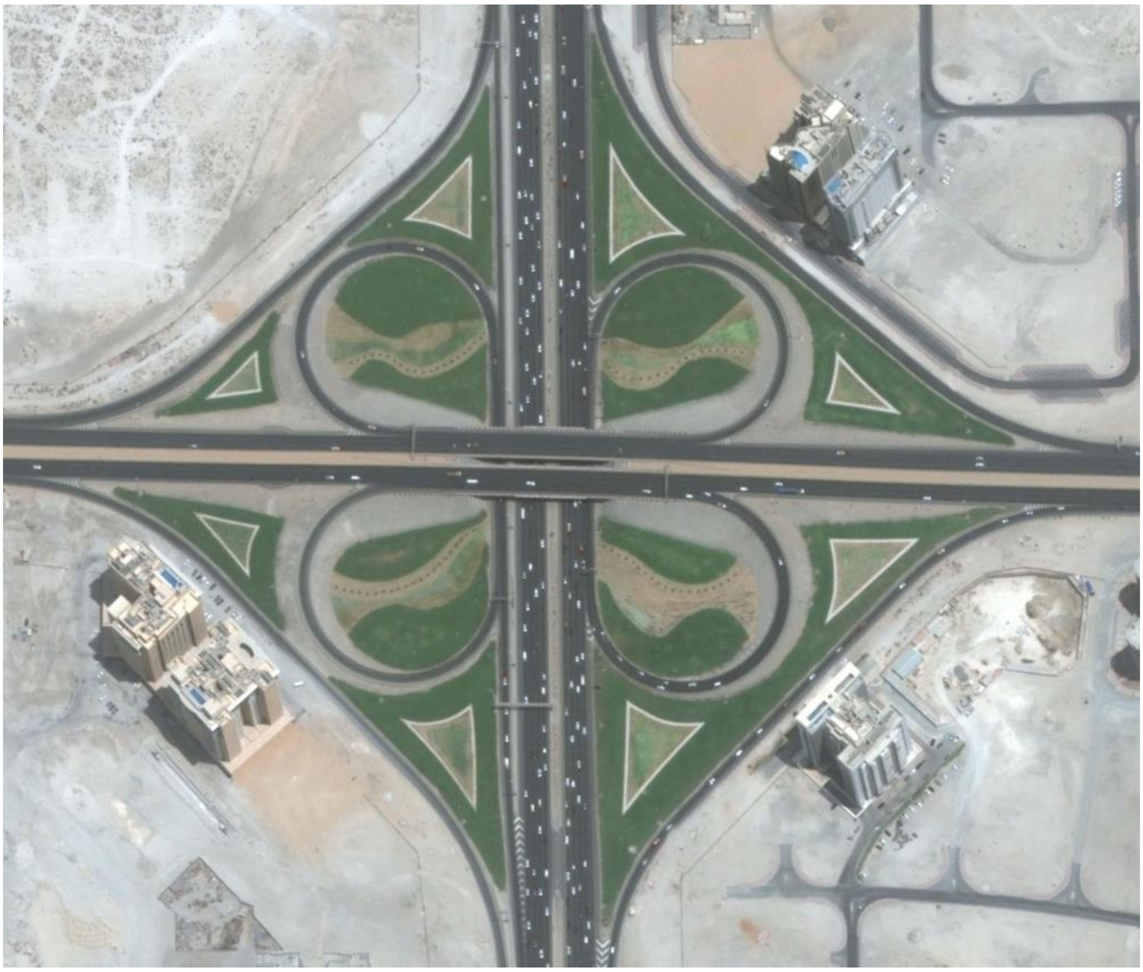


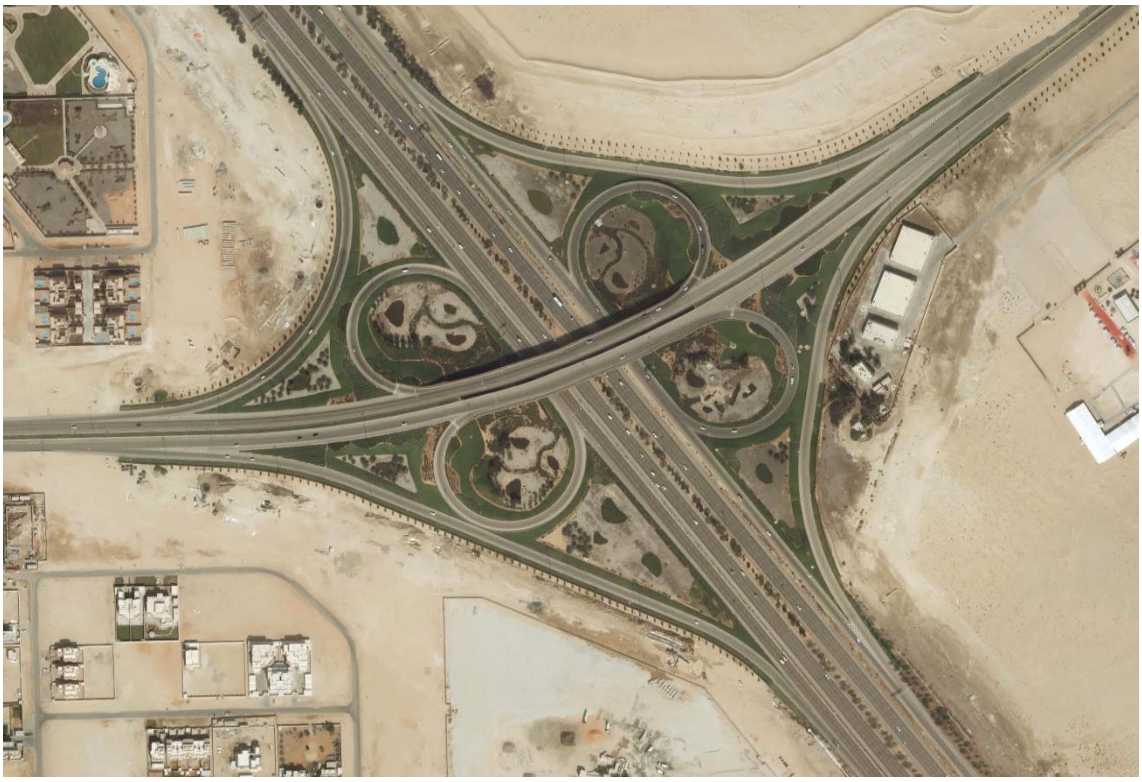
(a) Conventional Cloverleaf



(b) Cloverleaf With C-D Roadways

Figure 12-22: Cloverleaf Interchanges





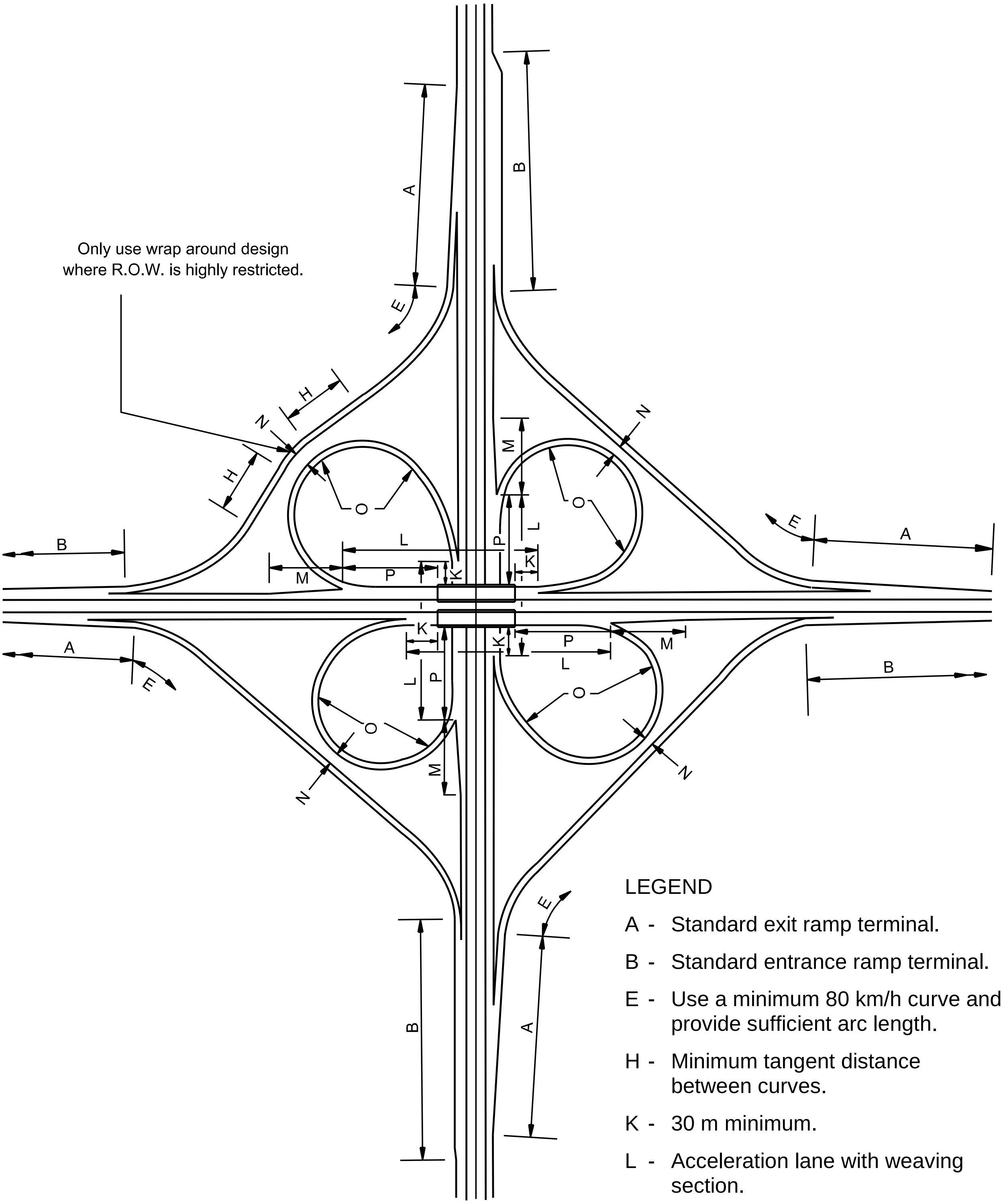
- Weaving sections between loop ramps must be made long enough to provide for satisfactory traffic operations.
- Where the crossroad is an expressway or other multilane roadway, considerable length of access control distance is needed along the crossroad to the first point of access.

12.4.6.2 Design Considerations

Figure 12-23 illustrates the design and layout of a typical cloverleaf interchange. In developing the cloverleaf interchange, consider the following steps:

1. Exit Gore. The first step is to locate the physical nose of the exit gore of the weaving section a minimum of three seconds of travel time at the design speed beyond the structure on each of the four interchange legs.
2. Weaving Section. The second step is to determine the minimum lengths required for the weaving sections. The weaving lengths are based on the design speed of the freeway and ramp curvature of the preceding entrance ramp. To relieve weaving problems, the minimum length of the weaving section must be determined using the *Highway Capacity Manual 2010* (17) and the appropriate level of service. Section 12.3.13 provides further guidance on weaving sections. If the proper weaving length cannot be provided, provide a collector-distributor roadway system to remove the weaving from the through lanes.
3. Entrance Gore. The location of the entrance gore is determined by adding the minimum weaving length to the exit gore nose location determined in Step 1. At a minimum, place the entrance gore 30 m before the structure and, desirably, 60 m before the structure.
4. Inner Loops. Once the physical noses of the exit and entrance gores have been determined, the horizontal alignment between the corresponding exit and entrance gores must be determined. Circular curve loop ramps are the most desirable geometrically because speeds and travel paths tend to be uniform. However, this is often impractical and compound curvature is generally required. The initial and final arcs of the loops may preclude using the specified radii for the design speed of the respective roadways and length of the weaving sections. A third intermediate arc is then compounded with initial and final arcs. If necessary, in obtuse quadrants, two arcs may be compounded between the initial and final curves. If the intermediate arc cannot be compounded with the minimum arc lengths provided in Section 12.5.7, one or both of the adjacent weaving sections containing the loop terminals must be adjusted and the process repeated.
5. Outer Connections. Once a satisfactory inner loop design has been developed, the designer must select the appropriate outer connection design. Desirably, this will be a tangent section connected by radii at the exit and entrance terminals. In place of the tangent section, compound curves having a radius greater than the radius preceding the exit terminal may be used. In urban areas where right-of-way may be restricted, a "wrap around" design may be used. In this situation, the central curve of the outer connector is normally made concentric to the arc at the centre of the inner loop and the selected radius should provide a minimum design speed of 60 km/h.

Figure 12-23: Cloverleaf Interchange Layout



LEGEND

A - Standard exit ramp terminal.

B - Standard entrance ramp terminal.

E - Use a minimum 80 km/h curve and provide sufficient arc length.

H - Minimum tangent distance between curves.

K - 30 m minimum.

L - Acceleration lane with weaving section.

M - Recovery taper.

N - Minimum 15 m.

O - Ramp radii compounded at 2:1.

P - Minimum 3 seconds travel time to physical nose of exit gore.

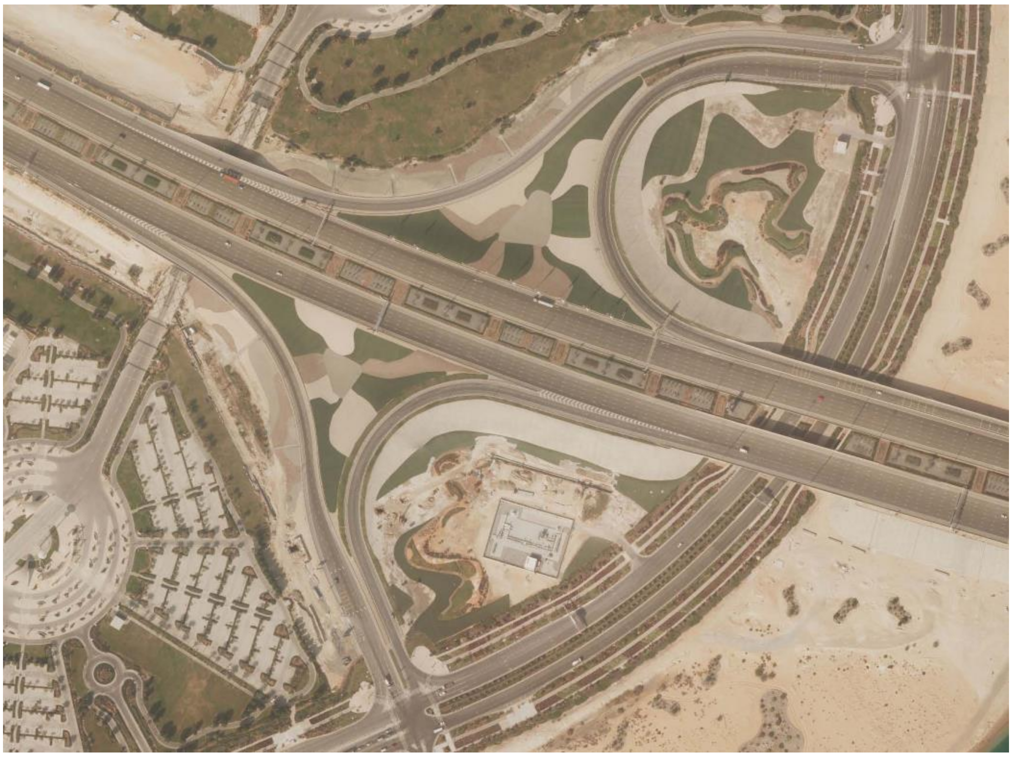
12.4.7 Partial Cloverleafs

12.4.7.1 General

Partial cloverleaf interchanges are those with loops in one, two, or three quadrants. Figure 12-24 shows several partial cloverleaf interchanges. Several of the disadvantages listed for full cloverleafs also apply to partial cloverleafs (e.g. geometric restriction of loops). However, some specific advantages of partial cloverleafs include:

- Partial cloverleafs provide access where one or more quadrants present adverse right-of-way and/or topographic problems that preclude a typical diamond interchange.
- Partial cloverleafs may accommodate heavy left-turn traffic by means of a loop and thereby improve capacity, operations, and safety.

Figure 12-24: Partial Cloverleaf Interchange



Depending upon site conditions, partial cloverleafs may offer the opportunity to increase weaving distances.

Partial cloverleaf designs can be segregated into the two-quadrant and four-quadrant partial cloverleaf. These are further explained as follows:

1. Two-Quadrant Partial Cloverleaf Interchanges. The two-quadrant partial cloverleaf interchange is normally used at those locations where cultural or natural features restrict the development of the diamond interchange. The two-quadrant partial cloverleaf interchanges Type A (Figure 12-25) and Type B (Figure 12-26) are used where right-of-way and/or construction is precluded in opposite quadrants of the interchange. The two-quadrant partial cloverleaf interchange Type C (Figure 12-27) is

used at intersections where additional structures or extensive relocation of the crossroad would be required to develop the diamond interchange (e.g. adjacent to major waterways, railroads).

The operations of the two-quadrant partial cloverleaf interchanges can be further defined as follows:

a. Type A. Both the exit and entrance terminals are located in advance of the structure and two channelized “T” intersections are formed on the crossroad. This arrangement reduces the probability of wrong-way movements. However, all turning movements from the crossroad must undergo a reverse operation; i.e. drivers travelling to the right must turn left and those travelling to the left must turn right.

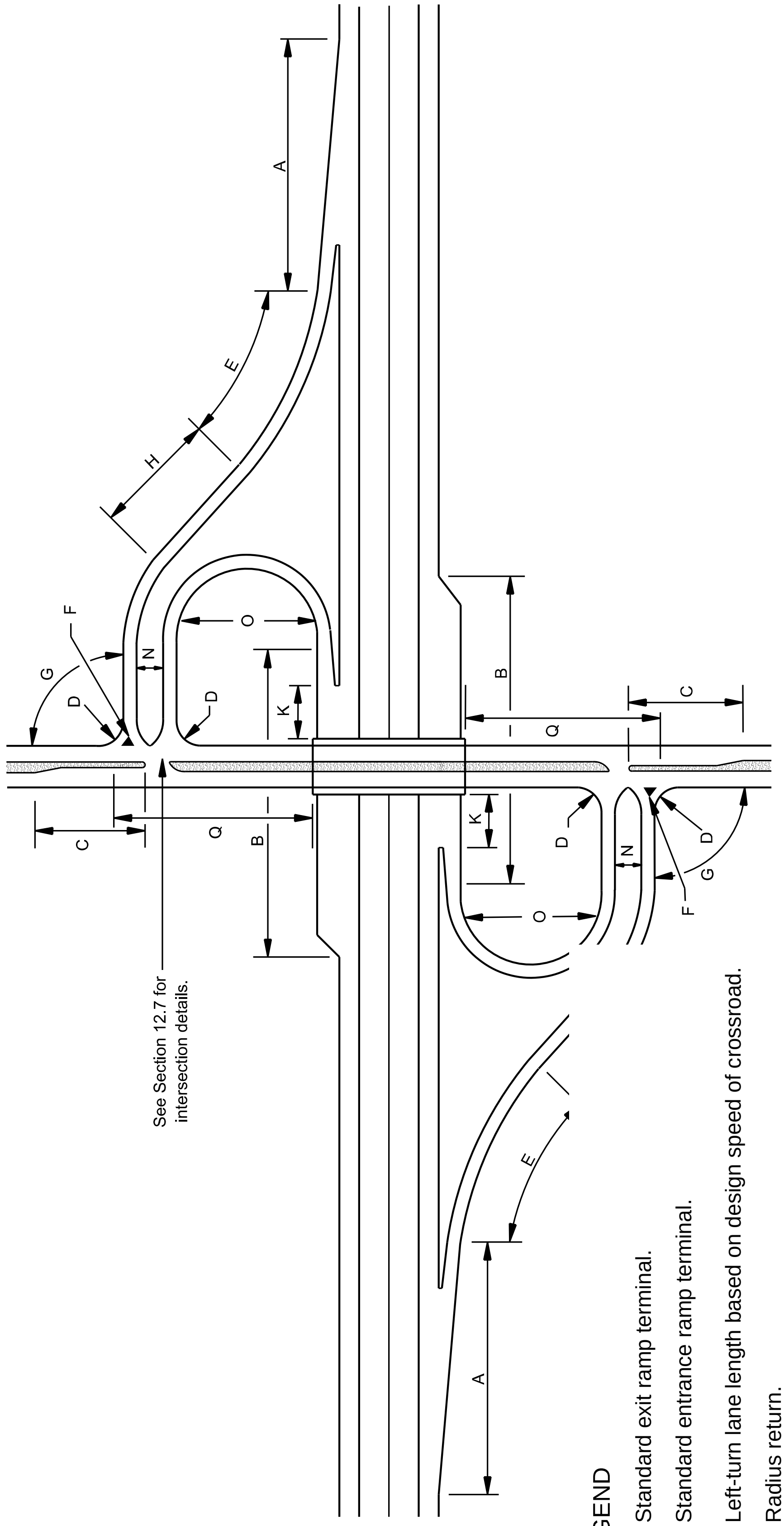
b. Type B. Because the “T” intersections allow normal operations for turning movements from the crossroad, the probability of wrong-way movements are greatly reduced. The exit terminals are located beyond the structure and, due to the lower design speed on the loop ramp; drivers tend to decelerate more on the mainline through lanes in advance of the exit.

c. Type C. No uniform pattern of operation is realised because traffic on the freeway exits in advance of the structure in one direction and beyond the structure in the other. Movements to the right or left from the crossroad are made by turning to the right for one direction and by turning left for the opposite direction. Consequently, channelization of the crossroad, with separate left-turn lanes is essential for proper operation.

2. Four-Quadrant Partial Cloverleaf Interchange. Figure 12-28 and Figure 12-29 illustrate the Type A and Type B four-quadrant partial cloverleaf interchanges, respectively. The four-quadrant partial cloverleaf interchange is used to provide for higher traffic volumes than the conventional diamond through the elimination of left-turning traffic at the crossroad ramp terminals.

Although there is a mixture of free-flowing and controlled terminals on the crossroad, there is little operational difficulty because the relative turning volumes and the arrangement of the ramp designs are compatible. Of the two types, Type A is probably more desirable, because it eliminates all conflicting left-turns from the crossroad.

Figure 12-25: Partial Cloverleaf Interchange Layout (Two Quadrants – Type A)

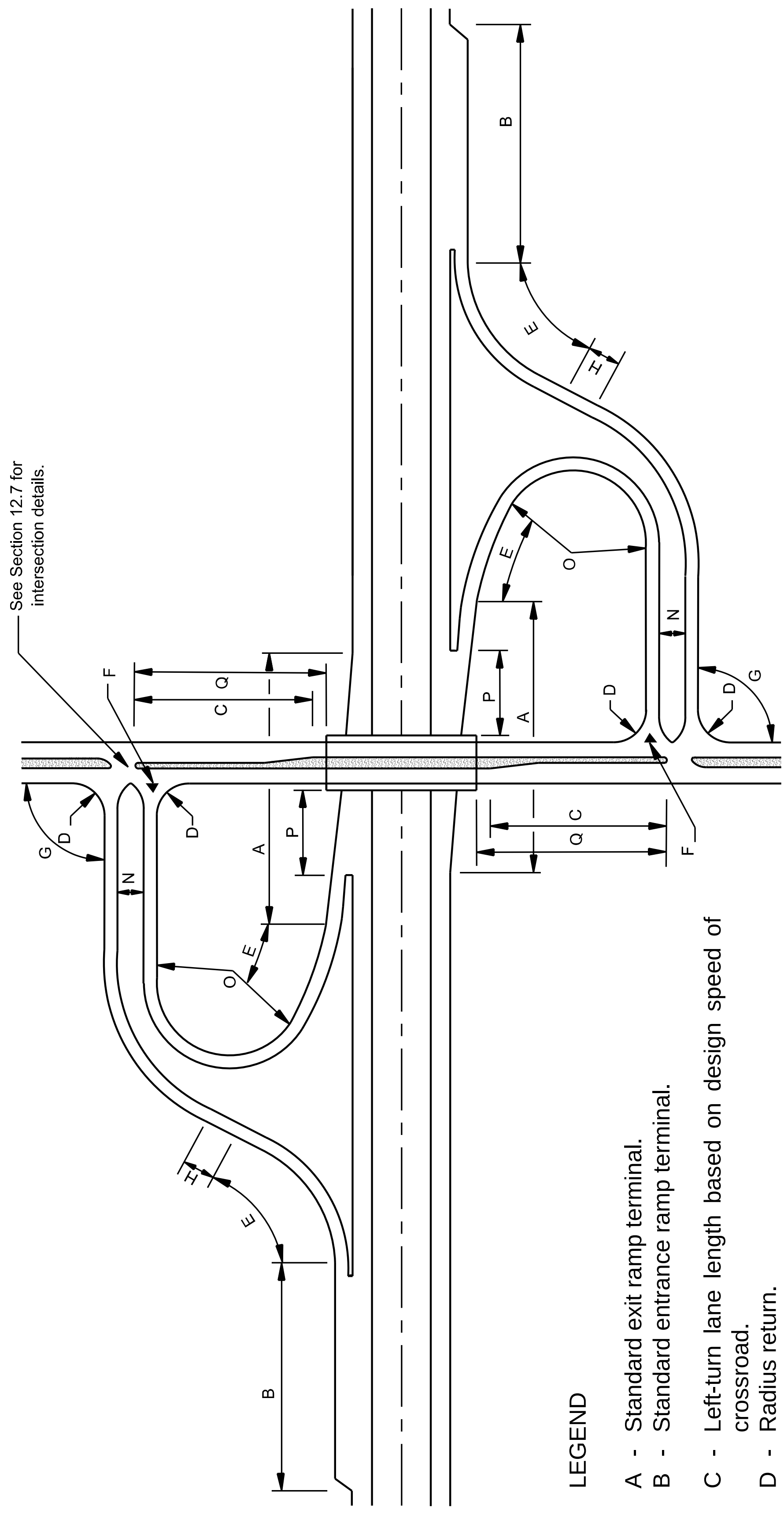


Note: Where the two ramps parallel each other, the elevations of the left edge of the outer ramp should approximately match the left edge elevations of the inner loop ramp.

LEGEND

- A - Standard exit ramp terminal.
- B - Standard entrance ramp terminal.
- C - Left-turn lane length based on design speed of crossroad.
- D - Radius return.
- E - Use a minimum 80 km/h curve and provide sufficient arc length.
- F - Directional corner island.
- G - Typically 90°.
- H - Minimum tangent distance between curves.
- K - 30 m minimum.
- N - Minimum 15 m.
- O - Ramp radii compounded at 2:1.
- Q - Minimum distance between structure and exit ramp based on intersection sight distance.

Figure 12-26: Partial Cloverleaf Interchange Layout (Two-Quadrant – Type B)



LEGEND

- A - Standard exit ramp terminal.
- B - Standard entrance ramp terminal.
- C - Left-turn lane length based on design speed of crossroad.
- D - Radius return.
- E - Use a minimum 80 km/h curve and provide sufficient arc length.
- F - Directional corner island.
- G - Typically 90°.
- H - Minimum tangent distance between curves.
- N - Minimum 15 m.
- O - Ramp radii compounded at 2:1.
- P - Minimum 3 seconds travel time at design speed of mainline to physical nose.
- Q - Minimum distance between structure and exit ramp based on intersection sight distance.

Figure 12-27: Partial Cloverleaf Interchange Layout (Two-Quadrant – Type C)

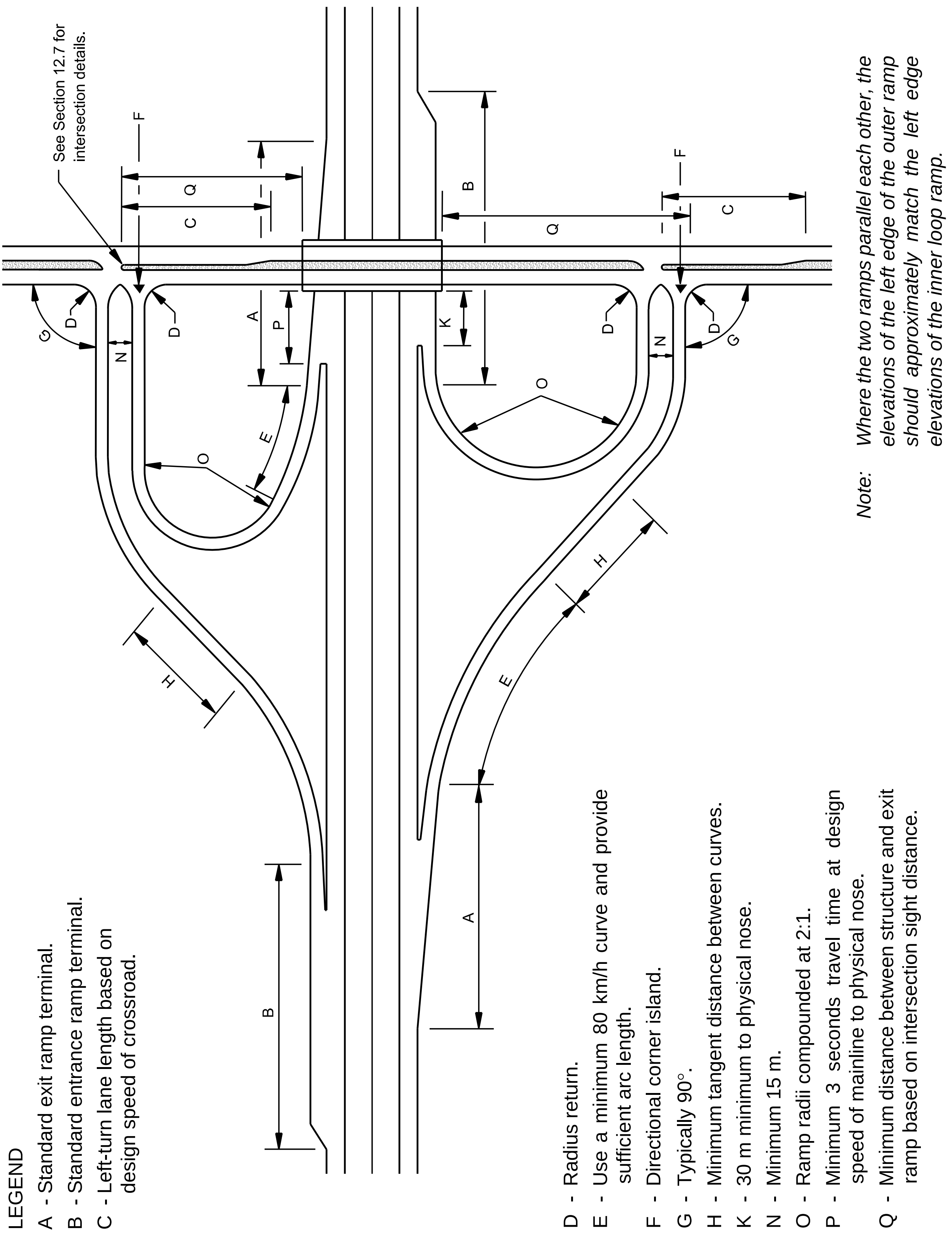
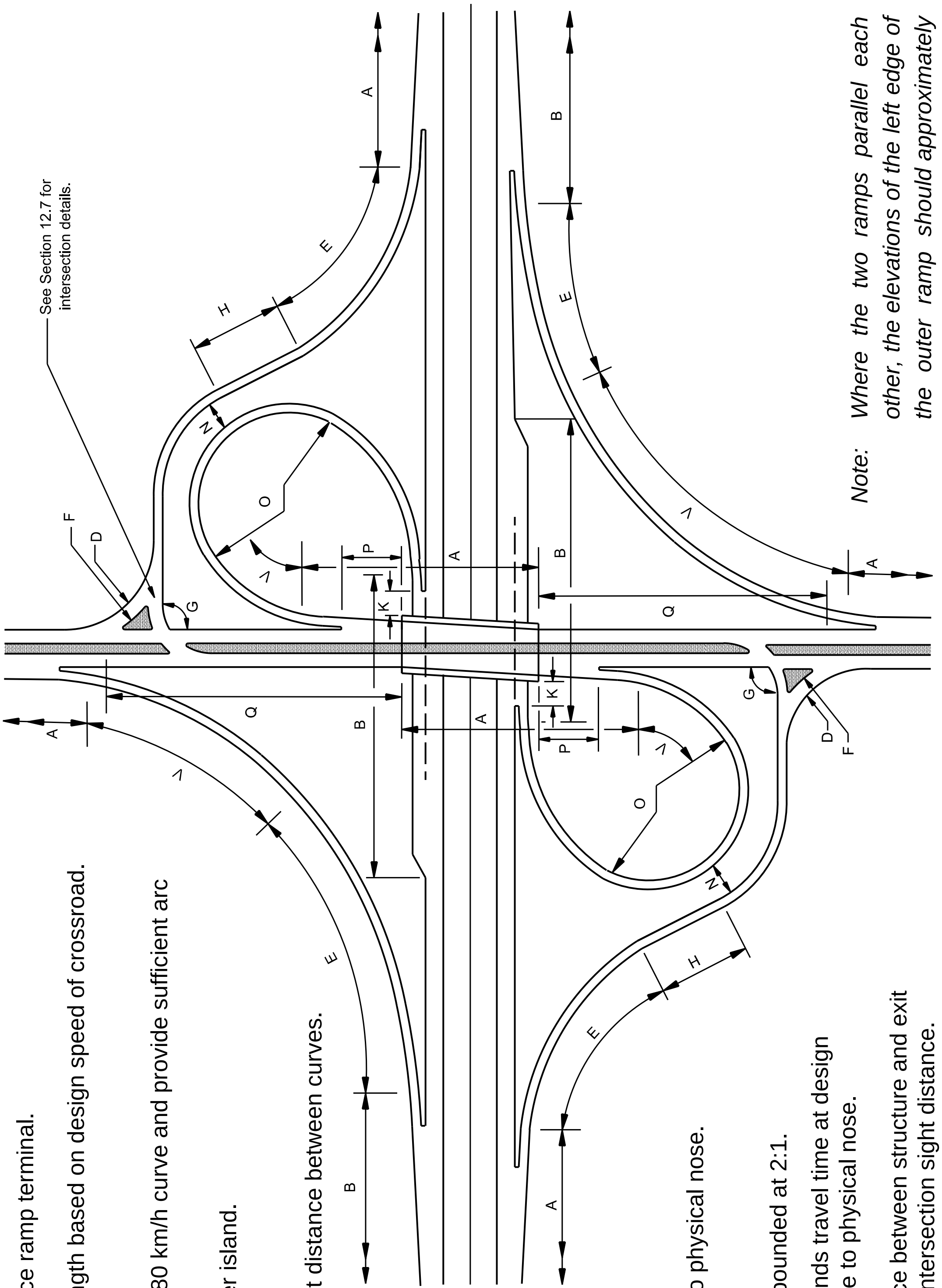


Figure 12-28: Partial Cloverleaf Interchange Layout (Four-Quadrant – Type A)

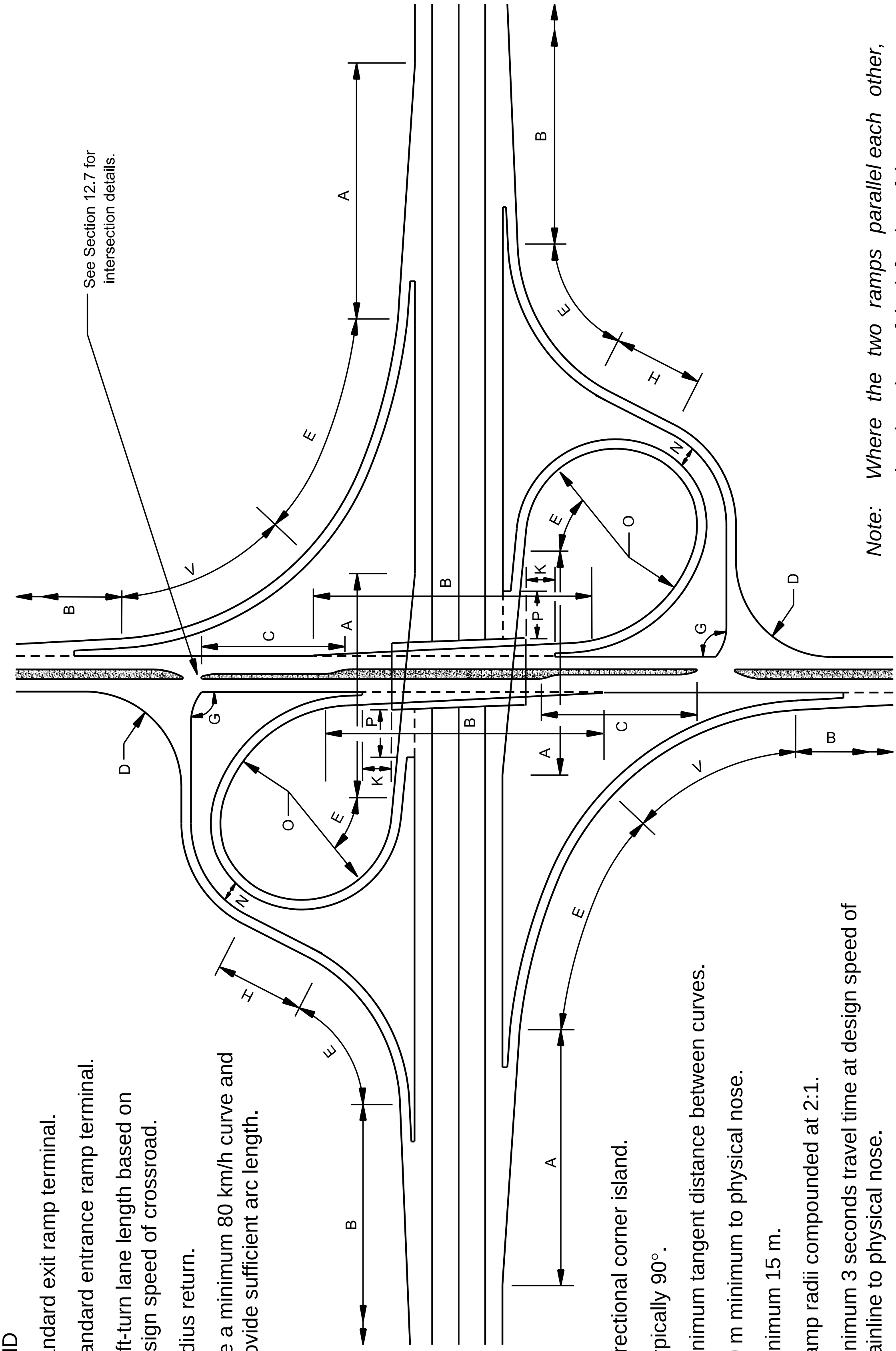


Note: Where the two ramps parallel each other, the elevations of the left edge of the outer ramp should approximately match the left edge elevations of the inner loop ramp.

LEGEND

- A - Standard exit ramp terminal.
- B - Standard entrance ramp terminal.
- C - Left-turn lane length based on design speed of crossroad.
- D - Radius return.
- E - Use a minimum 80 km/h curve and provide sufficient arc length.
- F - Directional corner island.
- G - Typically 90°.
- H - Minimum tangent distance between curves.
- K - 30 m minimum to physical nose.
- N - Minimum 15 m.
- O - Ramp radii compounded at 2:1.
- P - Minimum 3 seconds travel time at design speed of mainline to physical nose.
- Q - Minimum distance between structure and exit ramp based on intersection sight distance.
- V - Curve not less than 20 km/h below crossroad design speed and provide sufficient arc length.

Figure 12-29: Partial Cloverleaf Interchange Layout (Four Quadrants – Type B)



Note: Where the two ramps parallel each other, the elevations of the left edge of the outer ramp should approximately match the left edge elevations of the inner loop ramp.

LEGEND

A - Standard exit ramp terminal.

B - Standard entrance ramp terminal.

C - Left-turn lane length based on design speed of crossroad.

D - Radius return.

E - Use a minimum 80 km/h curve and provide sufficient arc length.

F - Directional corner island.

G - Typically 90°.

H - Minimum tangent distance between curves.

K - 30 m minimum to physical nose.

N - Minimum 15 m.

O - Ramp radii compounded at 2:1.

P - Minimum 3 seconds travel time at design speed of mainline to physical nose.

Q - Minimum distance between structure and exit ramp based on intersection sight distance.

V - Curve not less than 20 km/h below crossroad design speed and provide sufficient arc length.

12.4.7.2 Two-Quadrant Partial Cloverleaf Interchange

Figure 12-25, Figure 12-26, and Figure 12-27 provide the general design and layout criteria for Type A, Type B, and Type C partial cloverleafs, respectively. During the layout and design of the two-quadrant partial cloverleaf, consider the following steps:

1. Ramp/Crossroad Intersection Location:

    a. Type A. Place the physical nose of the standard entrance terminal a minimum of 30 m upstream from the structure. Design the smallest loop radius with a 50 km/h or 60 km/h design speed in rural areas and 40 km/h or 50 km/h design speed in urban areas. Project a tangent line from the loop radius to intersect with the crossroad at approximately 90 degrees. This procedure sets the location of both ramps and the location for the beginning of the left-turn lane on the crossroad. Check the intersection sight distance at the intersection of the exit ramp with the crossroad back to the left along the crossroad; see Section 4.6.

    b. Type B. Place the physical nose of the exit terminal three seconds of travel time beyond the structure. Design the smallest loop radius with a 50 km/h or 60 km/h design speed in rural areas and 40 km/h or 50 km/h design speed in urban areas. Project a tangent line from the loop radius to intersect with the crossroad at approximately 90 degrees. Layout the location of the left-turn lane on the crossroad to fit into the intersection of the entrance ramp. Check the intersection sight distance at the intersection of the exit ramp with the crossroad back to the right along the crossroad; see Section 4.6.

    c. Type C. The ramp/crossroad intersections for a Type C partial cloverleaf are located in the same manner as the respective terminals for Types A and B.

2. Loop Ramps. For Type A partial cloverleafs, the radii of succeeding arcs should increase in the direction of travel so that the traffic may enter the mainline roadway at a reasonably high operating speed. For Type B partial cloverleafs, the radii of succeeding arcs should decrease at a ratio of 2:1, with the arc of the sharpest curve being located immediately before the tangent section of the ramp.

3. Outer Connections. The tangent portions of the outer connectors are set parallel to the tangent portions of the loop ramps and are separated by a 15-m median. This width provides for a suitable common drainage section and minimises headlight glare from opposing traffic. The remaining portion of the outer connection is developed concentric with the loop ramp and then follows a line approximately 45 degrees in relationship to the mainline. The intervening tangent length between the reverse curves should be no less than the sum of 67% of the two superelevation runoff lengths.

12.4.7.3 Four-Quadrant Partial Cloverleaf Interchange

Figure 12-28 and Figure 12-29 illustrate the typical design and layout criteria for four-quadrant partial cloverleaf interchange Types A and B. The design procedures for the four-quadrant partial cloverleaf are similar to those for the cloverleaf and the corresponding two-quadrant partial cloverleaf. The loop ramps are designed in the same manner as those of the cloverleaf. However, because they use the standard entrance and exit terminals rather than the weaving section terminals, the loops are smaller than the conventional cloverleaf loops. The outer connectors are designed in the same manner as those of the two-quadrant partial cloverleaf interchange.

Provide a common drainage section between the outer connectors and the free-flow loops and set the tangent approach to the crossroad to intersect the crossroad at approximately 90 degrees. The right-turn free-flow direct ramps located in opposite quadrants consist of compound circular arcs where the adjacent radii should not exceed a ratio of 2:1. The standard exit and entrance terminals of the direct ramps are located a certain minimum distance from the intersection of the outer connections with the crossroad.

12.4.8 Trumpet Interchange

12.4.8.1 General

The trumpet type interchanges, illustrated in Figure 12-30, Figure 12-31, and Figure 12-32 are examples of three-leg interchanges where three of the turning movements are accommodated with direct or semi-direct connection ramps and one movement by a loop ramp.

Trumpet Types A and B are used primarily:

- at intersections with non-freeway spur connections or routes which are terminated at the freeway,
- at intersections with other roads that are contiguous with the freeway for a short distance and then diverge on their own alignment, or
- where future expansion to the unused quadrant is not practical or likely.

They are typically limited to intermediate traffic volumes that can be accommodated by single-lane ramps.

Figure 12-30: Trumpet Interchange

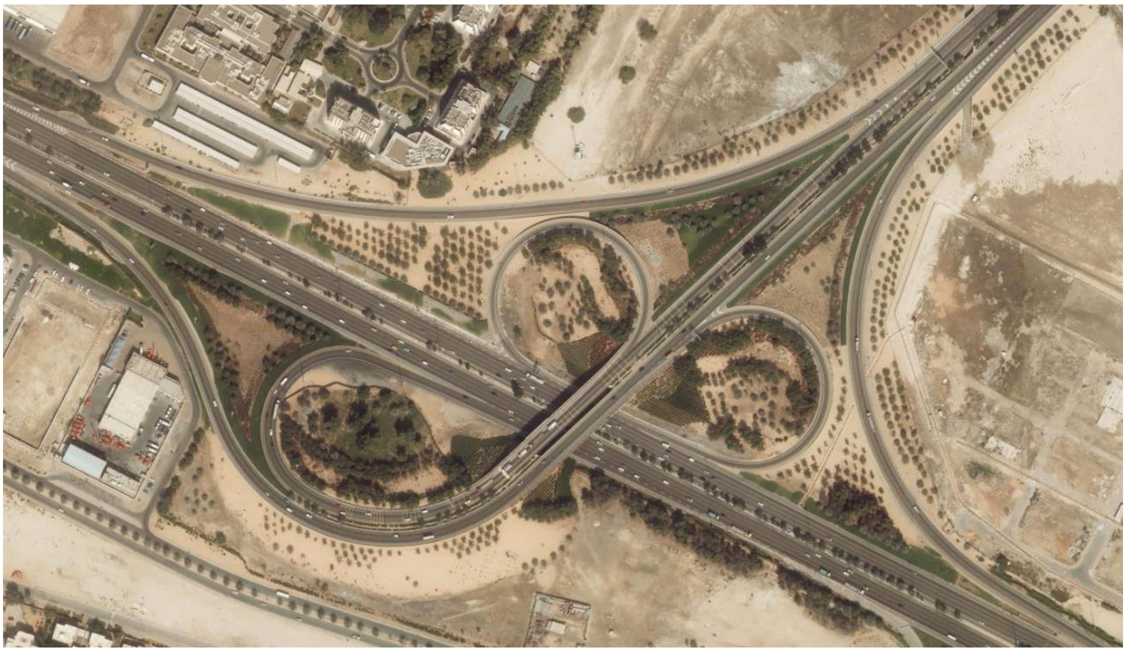


Figure 12-31: Trumpet Interchange Layout (Type A)

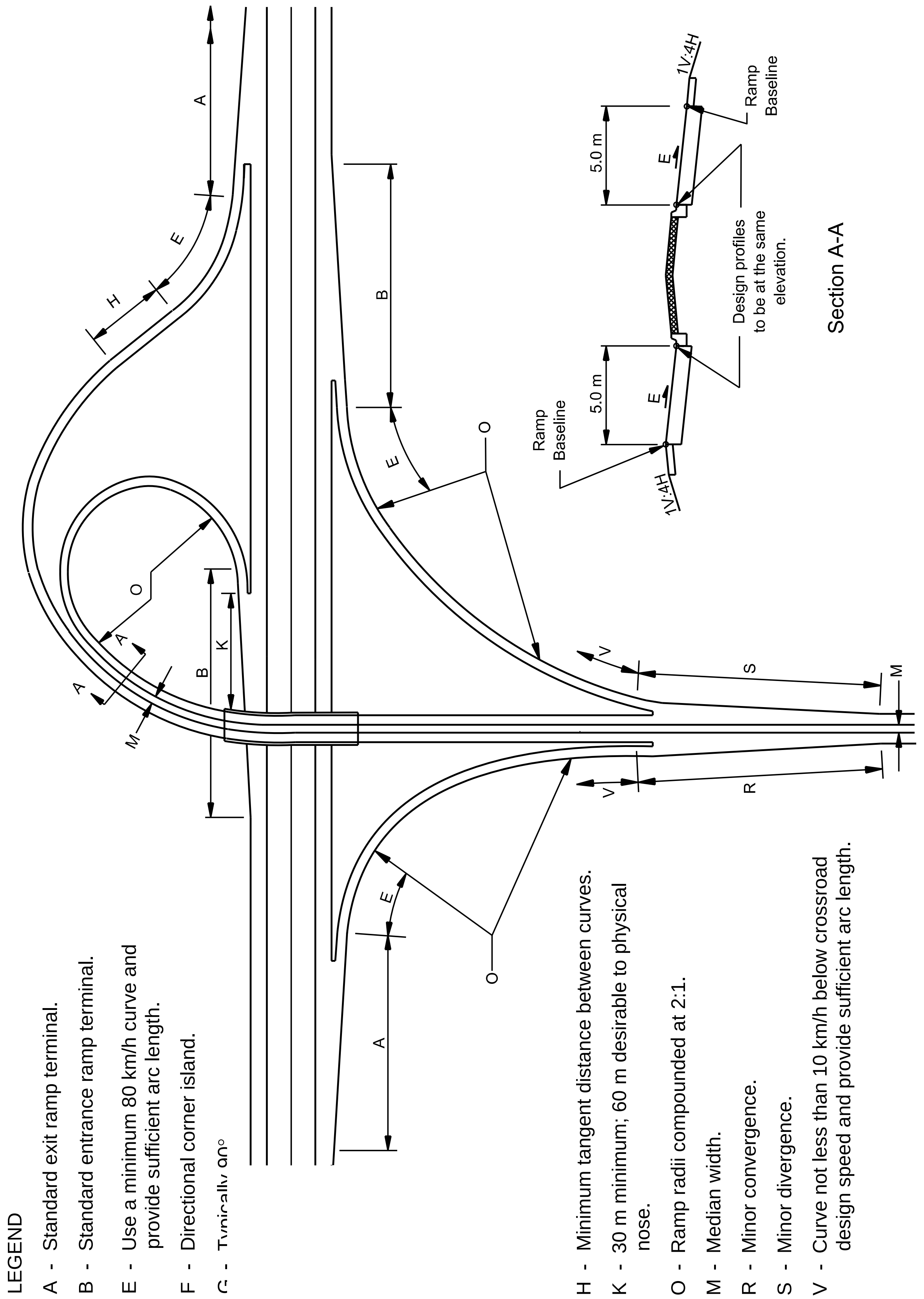
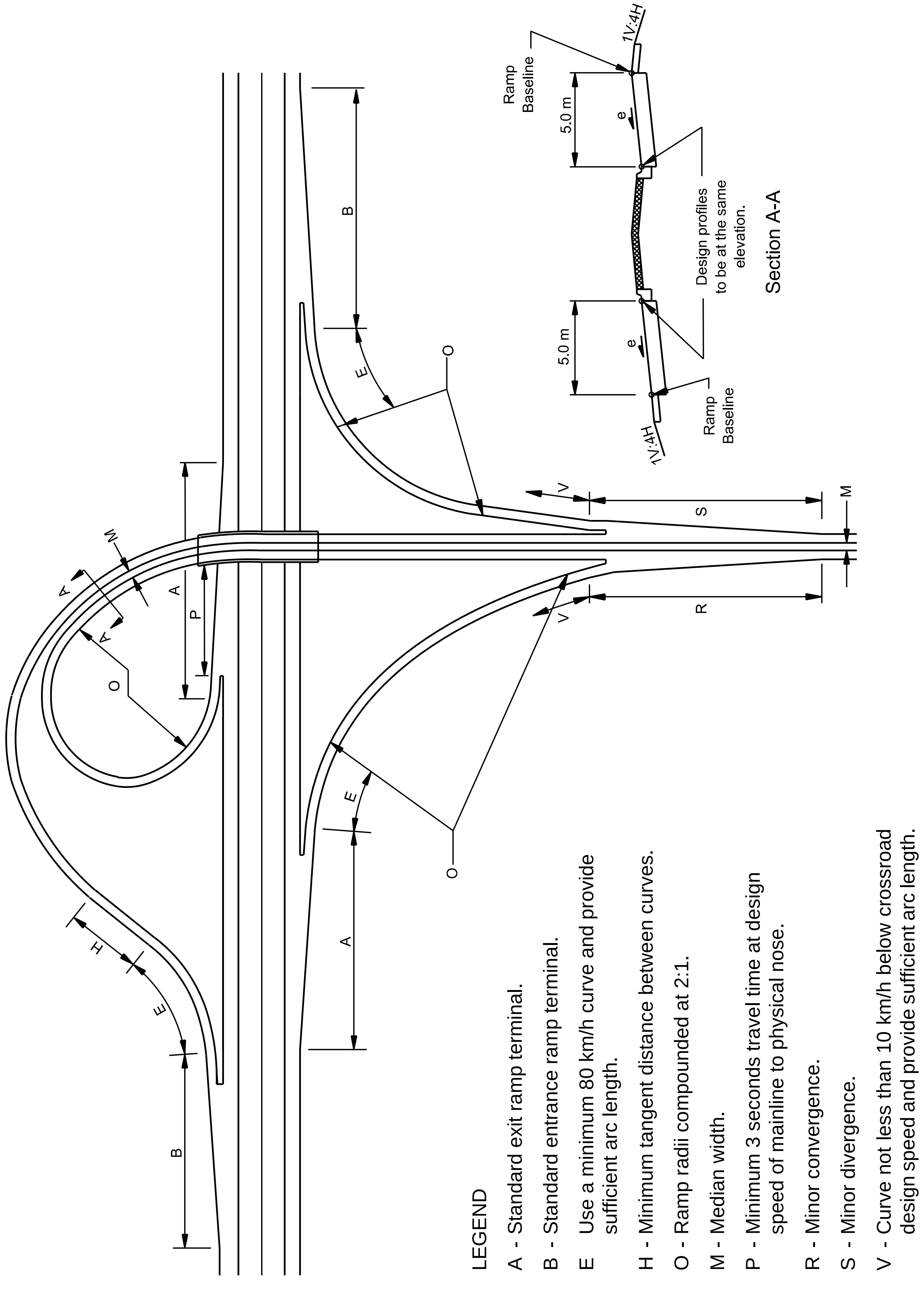


Figure 12-32: Trumpet Interchange Layout (Type B)



The “bell” of the trumpet is normally oriented to favour the predominant turning movements. Where the volume of traffic exiting from the freeway exceeds the volume entering from the minor road, use the trumpet Type A. Where the volume entering from the minor road exceeds the volume exiting from the freeway, use the trumpet Type B. Where the entering and exiting volumes are comparable, the trumpet Type A is preferred due to its better operational characteristics.

12.4.8.2 Design Considerations

Figure 12-31 and Figure 12-32 illustrate the design and layout criteria for Type A and Type B trumpet interchanges, respectively. Use Type A configuration where the predominate movement is left-turns from the freeway to the minor road. Use Type B configuration where the predominate movement is right turns from the freeway to the minor road.

The third leg of the Type A trumpet interchange approaching the primary roadway is on a curve. This is important information for the driver on the third leg approaching the loop ramp (seeing the curve), because loop itself may be hidden from view because of the bridge (freeway over) or a crest vertical curve (freeway under). The jug-handle ramp exit from the freeway has a larger radius and consequently a higher design/operating speed.

The Type B trumpet interchange design has a similar appearance in that loop ramp geometry is laid over, where the approaching roadway is on a curve visible to the driver to facilitate the operation of the jug-handle ramp. In addition, the exit gore for the loop ramp exit has been moved in advance of the bridge, improving two operational characteristics:

- the exit gore can be seen well in advance by the driver; and
- there is a portion of the ramp beyond the exit gore, but before the loop to allow adequate deceleration and speed transition by the driver off the freeway proper before negotiating the small radius of the loop itself.

In designing the trumpet interchange, first develop the location of the loop ramp and structure. This requires a certain amount of trial and error because the loop is a continuation of the minor road rather than a connection to a standard entrance and exit terminal. The loop and the outer connection are placed on curved alignment as they pass over the major road.

The outer connector is located adjacent and parallel to the roadway of the loop ramp and the rest of the ramp is located based on the selected design speed. These direct ramps provide turning movements to and from the minor road by using standard entrance and exit terminals on the freeway and minor convergence and divergence terminals on the minor road. Design the directional ramps for right turns to and from the minor road using compound curves and a minimum design speed of 80 km/h.

As shown in Figure 12-31 and Figure 12-32, the loop ramps or outer connectors do not exit from the minor road with the standard terminals, but are a continuation of the single-lane roadways. Because of sight distance restrictions, due to the presence of the structure or piers and higher possible speeds on the stem approach, motorists may be confronted with an abrupt transition in speed and alignment immediately beyond the structure. To minimise these operational difficulties, it is desirable to place the ramps on curved alignment with larger radii before passing over or under the freeway.

12.4.9 Direct and Semi-Direct Interchanges

Direct or semi-direct connection ramps are used for heavy left-turn movements, to reduce travel distance, to increase speed and capacity, and to eliminate weaving. These types of connections allow an interchange to operate at a better level of service than is possible with loops. Figure 12-33, Figure 12-34, Figure 12-35, and Figure 12-36 illustrate direct and semi-direct connection ramps arrangements.

Direct connection ramps do not deviate greatly from the intended direction of travel. Interchanges that use direct connections for the major left-turn movements are direct interchanges. This is true even if the minor left-turn movements are semi-direct connections (e.g. loop ramps).

Semi-direct connection ramps are where the motorist exits to the right first, heading away from the intended direction of travel, gradually reversing, and then passing around other interchange ramps before entering the other roadway.

Directional interchanges are most often provided in urban areas at freeway-to-freeway or freeway-to-arterial intersections. In rural areas, there is generally an insufficient traffic volume to justify the use of direct or semi-direct connection ramps in all quadrants. A direct interchange provides the highest possible capacity and level of service, but it is often costly to construct due to the number of structures required and amount of embankment.

No uniform design procedures can be established for direct or semi-direct connection ramps at interchanges due to the great variety of configurations. Loop ramps and weaving sections, where used, are designed as discussed in Sections 12.4.6 and 12.4.7. Because motorists perceive that higher operating speeds are possible on direct and semi-direct roadways, the alignment of these facilities should be as free flowing as practical.

12.4.10 Other Interchange Types

12.4.10.1 Hybrid Interchange

Hybrid interchanges use a mixture of interchange types and are commonly used. Their construction can consist of multiple interchange designs (e.g. loop ramps, flyovers and roundabouts). Figure 12-37 illustrates the combinations of a diamond and roundabout interchange.

12.4.10.2 Rotary Interchange (Roundabout Interchange)

A rotary or roundabout interchange, shown in Figure 12-38, is a normal roundabout except one (two-level) or both (three-level) mainlines pass under or over the whole interchange. The ramps of the interchanging highways meet at a roundabout or rotary on a separated level above, below, or in the middle of the two roadways.

Roundabout interchanges are much more economical in use of materials and land than other interchange designs, as the junction does not normally require more than three bridges to be constructed. However, their capacity is limited when compared to other interchanges and can become congested easily with high traffic volumes.

Figure 12-33: Directional Interchanges

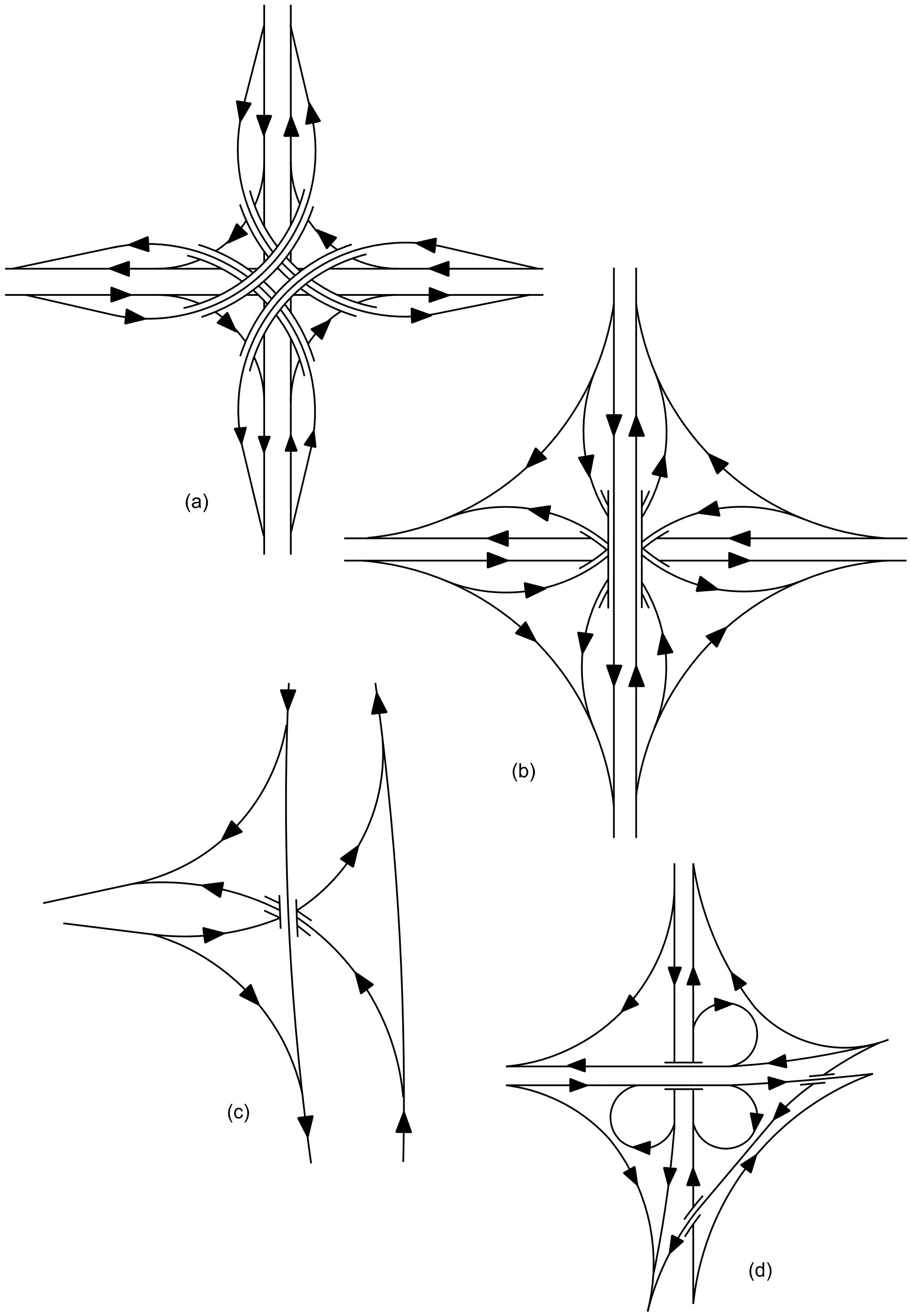
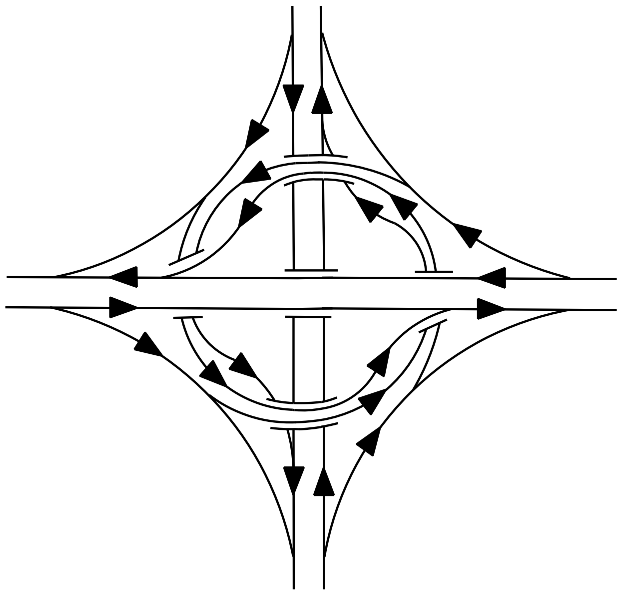
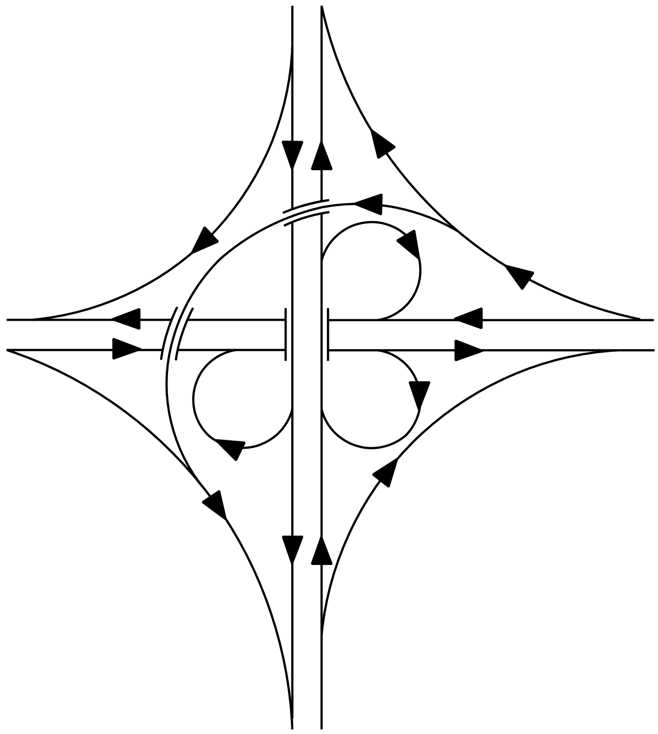


Figure 12-34: Directional Interchange with Semi Direct Connections



(a)



(b)

Figure 12-35: Directional Interchange

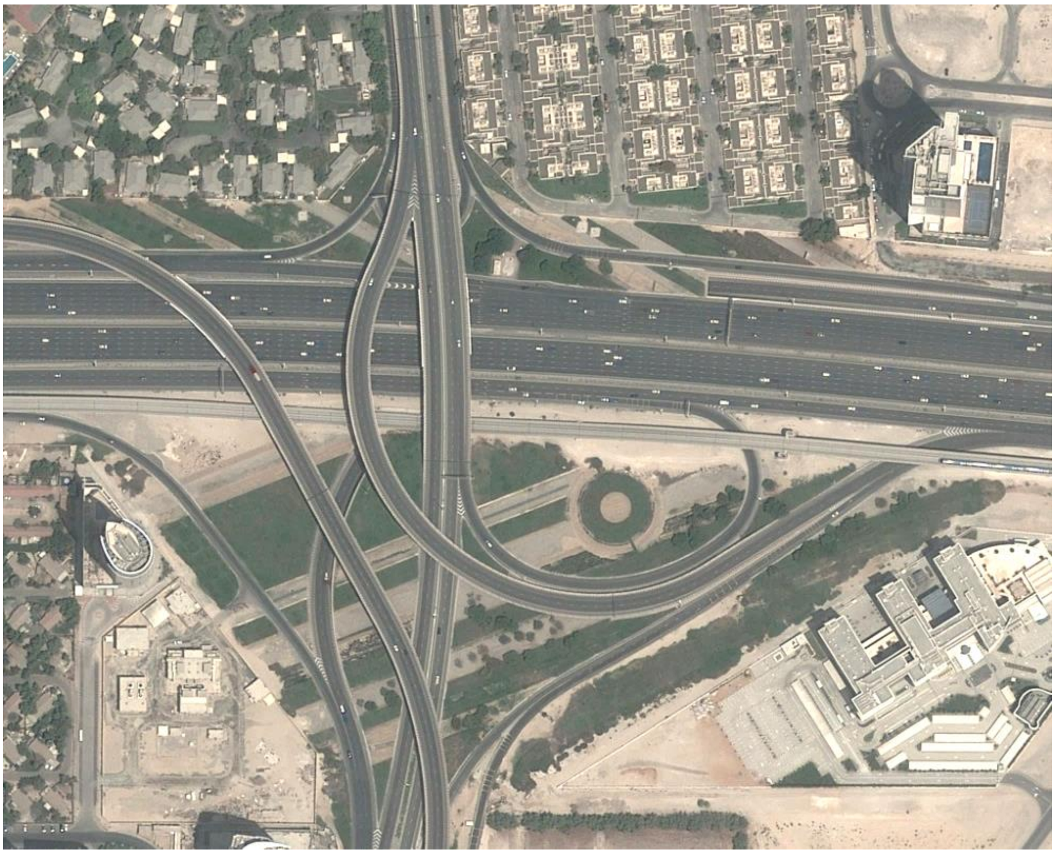


Figure 12-36: Directional Interchange with Semi Direct Connections

Figure 12-37: Diamond/Roundabout Interchange

Figure 12-38: Rotary (Roundabout) Interchanges

12.4.10.3 Turbine Interchange

Another alternative to the four-level stack interchange is the turbine interchange (also known as a whirlpool). The turbine/whirlpool interchange requires fewer levels (usually two or three) while retaining semi-direct ramps throughout, and has its left-turning ramps sweep around the centre of the interchange in a spiral pattern in right-hand driving. See Figure 12-39 and Figure 12-40.

Turbine interchanges offer slightly less vehicular capacity because the ramps typically turn more often and change height quicker. They also require more land to construct than the typical four-level stack interchange.

In areas with rolling or mountainous terrain, turbine interchanges can take advantage of the natural topography of the land due to the constant change in the height of their ramps, and hence these are commonly used in the areas where conditions apply, reducing construction costs compared to turbine interchanges built on level ground.

Figure 12-39: Two-Level Turbine Interchange

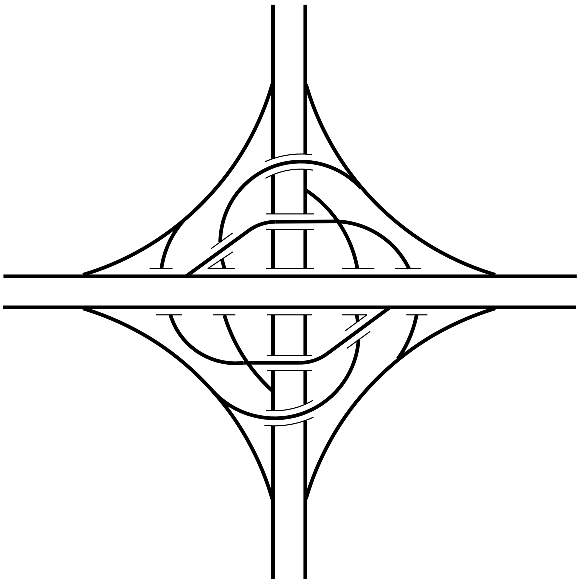
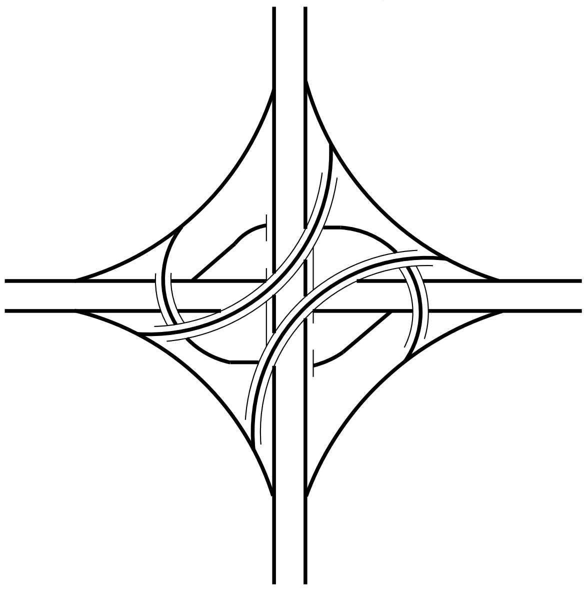


Figure 12-40: Three-Level Turbine Interchange

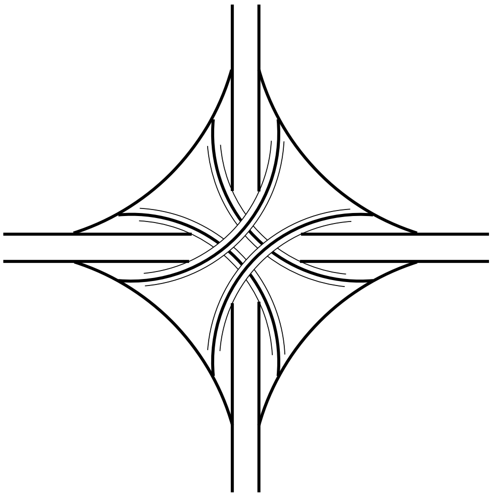


12.4.10.4 Stack Interchange

A stack interchange is a four-way interchange whereby left turns are handled by direct flyover/under ramps. See Figure 12-41. To go left, vehicles first turn slightly right, on a right-turn off-ramp, to exit, then complete the turn via a ramp that crosses both highways, eventually merging with the right-turn on-ramp traffic from the opposite quadrant of the interchange. A stack interchange, then, has two pairs of left-turning ramps, of which can be stacked in various configurations above or below the two interchanging highways.

Stacks do not suffer from the problem of weaving, but require massive construction work for their flyovers. A standard stack interchange includes roads on four levels. This is significantly more expensive than other interchanges, and additionally may suffer from objections of local residents, because of their high visual impact. Large stacks with multiple levels may have a complex appearance and are often colloquially described as mixing bowls, mixmasters, or as spaghetti bowls or spaghetti junctions.

Figure 12-41: Stack Interchange



12.4.10.5 Clover Stack Interchange

The partial cloverleaf interchange (parclo) design modified for freeway traffic has emerged over time, eventually leading to the cloverstack interchange. See Figure 12-42 and Figure 12-43

Ramps are longer to allow for higher ramp speeds, and loop ramp radii are made larger as well. The large loop ramps eliminate the need for a fourth, and sometimes a third level in a typical stack interchange, as only two directions of travel use flyover/under ramps.

Cloverstacks are cheaper to build than stack interchanges and are less of an eyesore for local residents. By using the loop ramps in opposite quadrants, weaving is also eliminated. However, cloverstacks require significant right-of-way to construct and the loop ramps are not as efficient as flyover/under ramps in terms of traffic flow. The cloverstack design is becoming more and more popular, and is commonly used to upgrade cloverleaf interchanges to increase their capacity and eliminate weaving.

Figure 12-42: Two-Level Clover Stack Interchange

Figure 12-43: Three-Level Clover Stack Interchange

12.4.10.6 Three-Leg Interchanges

An interchange with three intersecting legs consists of one or more highway grade separations and on-way roadways for all traffic movements. When two of the tree intersection legs form a through road and the angle of intersection is not acute, the term “T-interchange” applies. When all three intersection legs have a through character of the intersection angle with the third intersection leg is small, the interchange may be considered a Y-interchange. These are further discussed as follows:

1. Y Interchange. A full Y-interchange is typically used when a three-way interchange is required for two or three highways interchanging in semi-parallel/perpendicular directions, but it can also be used in right-angle case as well. See Figure 12-44. Their connecting ramps can spur from either the right or left side of the highway, depending on the direction of travel and the angle.

   Full Y-interchanges use flyover/flyunder ramps for both connecting and mainline segments, and they require a moderate right-of-way and moderate costs because only two levels or roadway are typically used. They get their name due to their resemblance to the capital letter Y, depending upon the angle from which the interchange is seen.

2. T-Interchange. A T-interchange uses flyover/under ramps in all directions at a three-way interchange, but some of the splits and merges are switched to avoid ramps to and from the passing lane. See Figure 12-45. T-interchanges are very efficient, but are expensive to build compared to other three-way interchanges. They also require three levels, which can be an

eyesore for local residents. However, the T-interchange is preferred to a trumpet interchange because a trumpet requires a loop ramp, which speeds can be reduced to as little as 40 km/h, but flyover ramps can handle much faster speeds.

Figure 12-44: Future Y Interchange

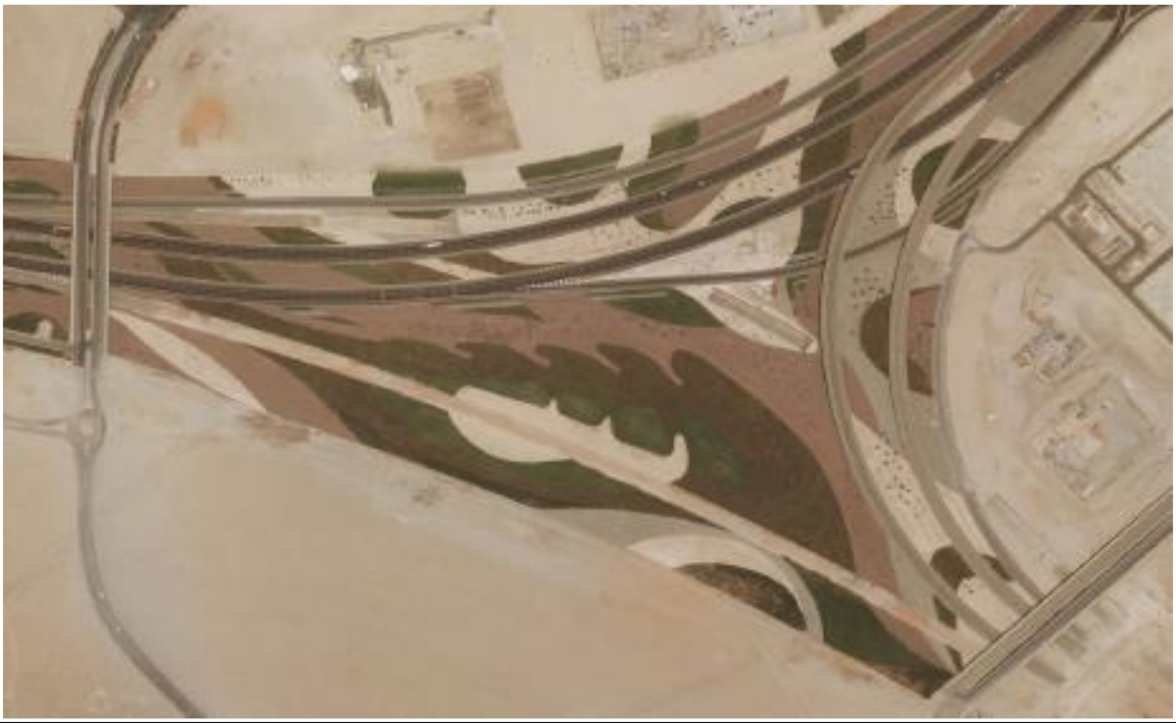
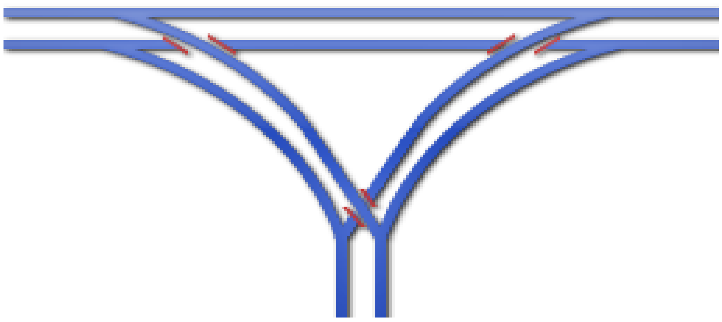


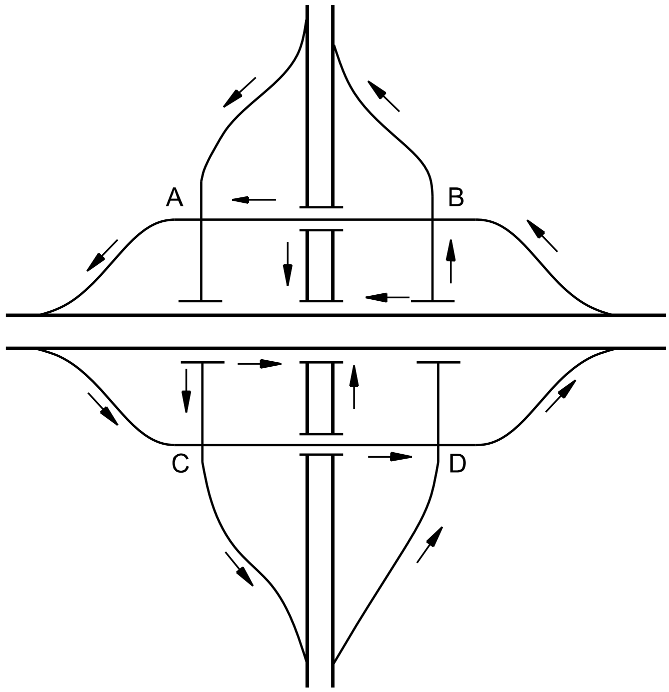
Figure 12-45: T-Interchange



12.4.10.7 Three Level Diamond/Volleyball Interchange

The three-level diamond/volleyball interchange is three levels high, and handles interchanging ramps via four at-grade intersections, points A, B, C, and D as shown in Figure 12-46. This interchange provides uninterrupted flow of through traffic on both of the intersecting highways. Only the left-turning movements cross at grade. This arrangement is applicable where the cross street carries large traffic volumes and topography is favourable. The right-of-way required is much less than other layouts with comparable capacity. The disadvantage of this interchange type is the turning movements must stop or slow down substantially. Signals are required at the at-grade intersections for high-volume situations. The signals are normally synchronized to provide continuous movements through a series of left turns once the area is entered.

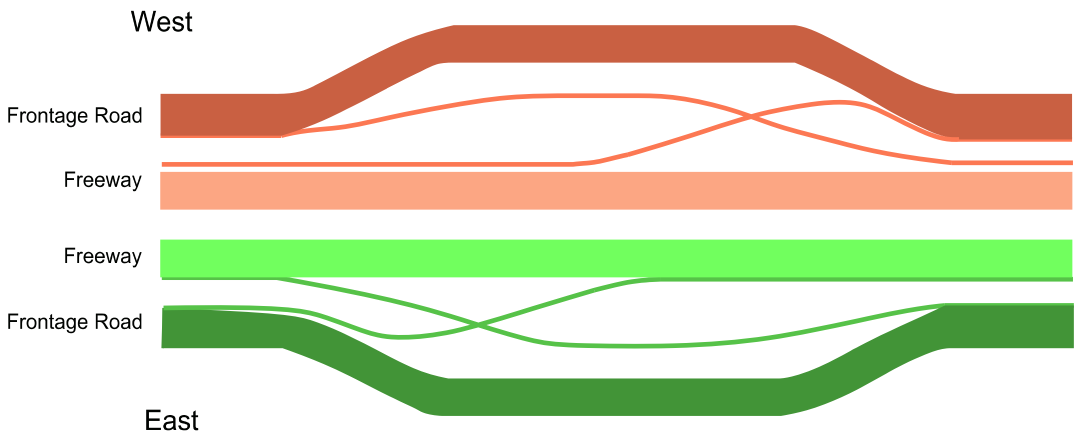
Figure 12-46: Three Level Diamond/Volleyball Interchange



12.4.10.8 Basket Weave Interchange

A basket weave interchange is commonly found on freeways using a collector/express system or long collector-distributor roadways; see Figure 12-47. In a basket weave, one roadway is able to interchange with itself, allowing traffic traveling in the same direction to switch between carriageways through the use of flyover/under ramps created between two carriageways without causing weaving. These interchanges usually involve left exits and entry for the outer carriageway (right in left-hand drive) but can be configured to meet on the right.

Figure 12-47: Basket Weave Interchanges



12.4.11 Selection

Typically, several interchange types should be evaluated for potential application considering the following:

- compatibility with the roadway system and functional classification of the intersecting road;
- route continuity and uniformity with adjacent interchanges;
- level of service for each interchange element (e.g. freeway ramp terminal, ramp proper, ramp/crossroad terminal);
- operational and safety considerations (e.g. signing);
- availability of access control along the crossroad;
- road-user impacts (e.g. travel distance and time, convenience, comfort);
- driver expectancy;
- topography and geometric design;
- right-of-way impacts and availability, construction and maintenance costs, and potential for stage construction;
- accommodation of pedestrians and cyclists on crossroad;
- environmental impacts; and
- potential growth of surrounding area.

Figure 12-1 depicts interchanges that are adaptable on freeways as related to classifications on intersecting facilities in rural and urban environments.

In addition, consider the following, which will influence the selection of an interchange type:

1. Basic Types. A freeway interchange will be one of two basic types. A system interchange will connect a freeway to a freeway; a service interchange will connect a freeway to a lesser facility.

2. Urban/Rural. In rural areas where interchanges occur relatively infrequently, the type selected is normally influenced by existing topography and environmental factors. In urban areas where restricted right-of-way and close spacing of interchanges are common, the type selection and design of the interchange may become more complex. The operational characteristics of the crossroad and proximity of nearby interchanges must be considered when selecting and designing an urban interchange.

3. Capacity. The need for loop ramps or other free-flowing ramps may depend upon the capacity of the ramp termini to adequately accommodate the turning traffic. Conduct a capacity analysis to determine if the ramp termini will be adequate and to determine the appropriate number of approach lanes on the crossroad and ramps. Section 12.6.3 provides further guidance on capacity analysis for ramp termini.

4. Movements. All interchanges should provide for all movements, even when the anticipated turning volume is low.

12.5 Ramp Design

12.5.1 Ramp Types

The components of a ramp include the freeway ramp terminal, the ramp proper, and a free-flow or controlled ramp terminal at the crossroad. Although ramps have varying shapes, each can be classified into one or more of the types illustrated in Figure 12-48 and discussed in the following sections.

12.5.1.1 Loop Ramps

There are two types of loop ramps:

1. Free-Flow. The free-flow loop, Figure 12-48(a), consists of compounded circular arcs that turn through approximately 270 degrees. The initial and final curves of the loop are tangent to the standard exit or entrance terminal or to a weaving section, depending upon the interchange type. The free-flow loop is a standard component of the cloverleaf interchange, the four-quadrant partial cloverleaf interchange, and the trumpet interchange. Free-flow loops are designed so that the central arc is a sharper radius than that of either the initial or final arcs, or the central arc is intermediate between the two. Motorists decelerate from the speed of the through roadway over the initial portion of the ramp and accelerate uniformly over the final portion of the ramp.

   Do not use loop ramps where the central arc has a greater radius than either the initial or the final arcs.

2. Controlled Terminal. Controlled terminal loops, Figure 12-48(b), are a component of the two-quadrant partial cloverleaf interchange. They are used most often with the standard entrance and exit terminals. Controlled terminals are provided at the intersections with the crossroad and permit both right- and left-turning movements. Wherever practical, design the angle of intersection for 90 degrees.

12.5.1.2 Diagonal Ramps

Diagonal ramps, Figure 12-48(c), are a component of the diamond interchange. Standard entrance and exit terminals are used on the major road, and controlled terminals are provided on the crossroad. The angle of intersection with the crossroad varies between 60 degrees and 90 degrees; see Section 12.4.2.2.

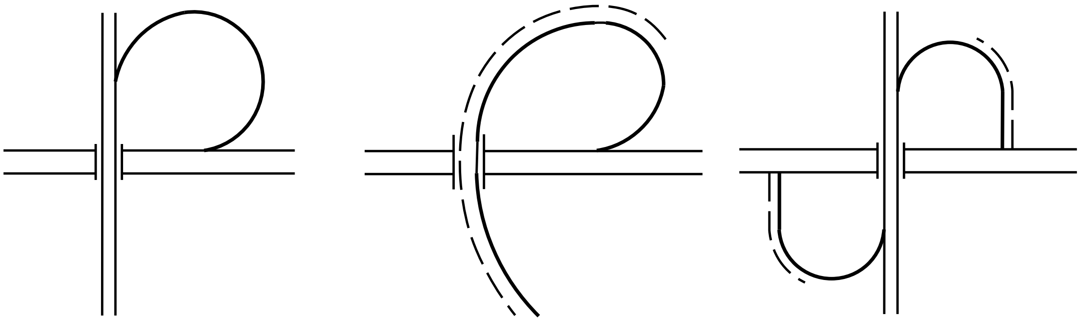
12.5.1.3 Outer-Connector Ramps

Outer-connector ramps are in the same quadrant and to the outside of loop ramps; see Figure 12-48(d). They may have free-flow operation (e.g. at cloverleaf or trumpet interchanges) or have controlled operations (e.g. at partial cloverleaf interchanges).

12.5.1.4 Semi-Direct Connection Ramps

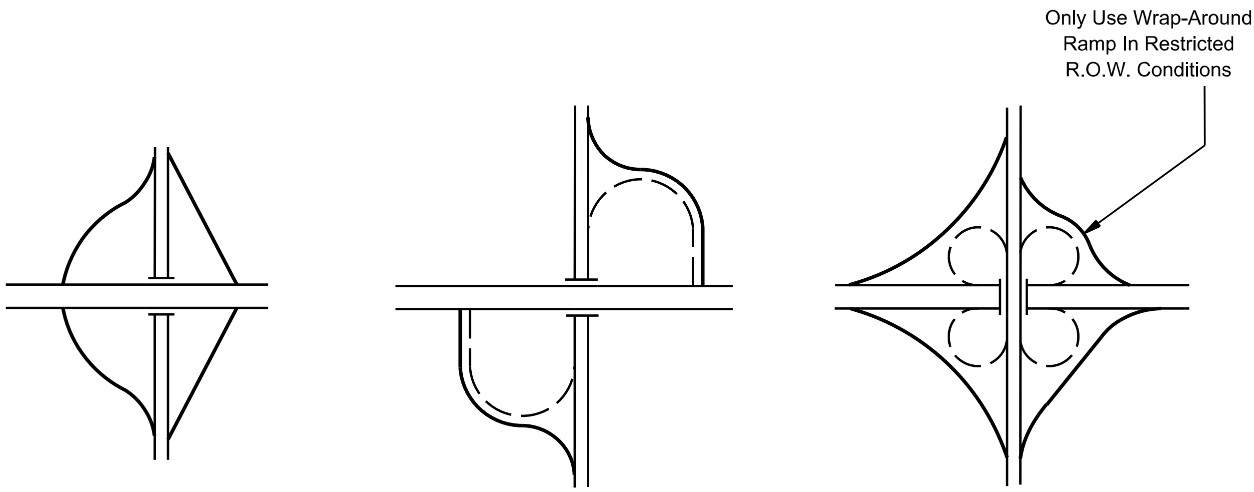
Semi-direct connection ramps are indirect in alignment, yet more direct than a loop ramp. These ramps are illustrated in Figure 12-48(e). Motorists making a left turn normally exit to the right and initially turn to the right, reversing direction before entering the intersecting road. The outer connection of the trumpet interchange is also a semi-direct connection ramp.

Figure 12-48: Ramp Types



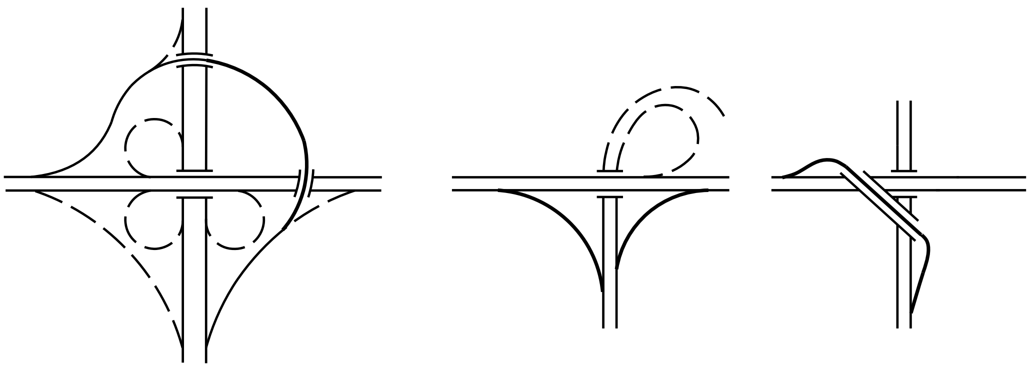
(a) Free-Flow Loop Ramps

(b) Controlled-Loop Ramps



(c) Diagonal Ramps

(d) Outer-Connection Ramps



(e) Semi-Direct Connection Ramp

(f) Direct Connection Ramps

Source: (1)

12.5.1.5 Direct Ramps

Direct connection ramps do not deviate greatly from the intended direction of travel. These are illustrated in Figure 12-48(f) as an element of a trumpet interchange. They are also used to accommodate single lane, right-turning traffic on four-quadrant partial cloverleafs, and direct interchanges.

12.5.2 Collector-Distributor Roadways

12.5.2.1 Usage

A collector-distributor (C-D) roadway is an auxiliary roadway parallel to and separated from the main travelled way that serves to collect and distribute traffic from several access points. It provides greater capacity and permits higher operating speeds to be maintained on the main travelled way. C-D roadways may be provided at single interchanges, through two adjacent interchanges or, in urban areas, continuously through several interchanges. The bottom illustrations in Figure 12-21 and Figure 12-22 are full cloverleaf interchanges with C-D roadways.

Usually, interchanges designed with single exits are superior to those with two exits, especially if one exit is a loop ramp or the second exit is a loop ramp preceded by a loop entrance ramp. Whether used in conjunction with a full cloverleaf or with a partial cloverleaf interchange, the single-exit design may improve the operational efficiency of the entire interchange. C-D roadways use the single exit approach to improve the interchange operational characteristics. C-D roadways will:

- remove weaving manoeuvres from the mainline and transfer them to the slower speed C-D roadways,
- provide high-speed single exits and entrances from and onto the mainline,
- satisfy driver expectancy by placing the exit before the grade separation structure,
- simplify signing and the driver decision-making process, and
- provide uniformity of exit patterns.

C-D roadways are most often warranted when traffic volumes (especially in weaving sections) are so high that the interchange cannot operate at an acceptable level of service. They also may be warranted where the speed relationship between weaving and non-weaving vehicles is significant.

12.5.2.2 Capacity Analyses

Capacity for C-D roadways can be determined using the same methodologies that apply to merge and diverge section and weaving sections. The designer should note the following:

1. <u>Weaving Analysis</u>. The procedure to determine the level of service (LOS) for weaving segments as discussed in Section 12.3.13 also apply to C-D roadways provided the free-flow speed is greater than or equal to 80 km/h. For lower free-flow speeds, the methodology discussed in the *Highway Capacity Manual 2010* (17) will produce an approximate result. Table 12-3 provides the LOS for weave segments, which are also applicable for C-D roadways.

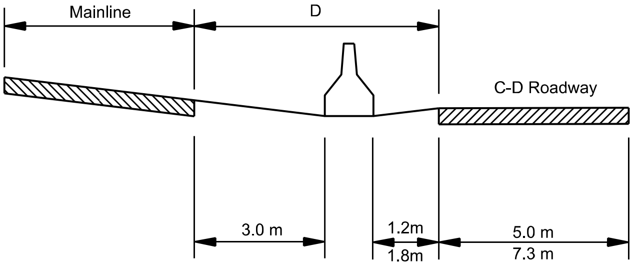
2. Merge/Diverge Analysis. The capacity discussion for ramp terminals as discussed in Section 12.6.3 provides an approximate capacity for C-D roadways. The LOS for ramp terminals are provided in Table 12-15.

12.5.2.3 Design

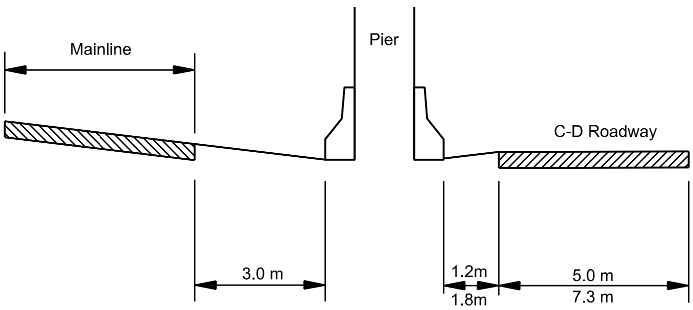
When designing C-D roadways, consider the following:

1. Design Speed. The design speed of a C-D roadway usually ranges from 60 km/h to 80 km/h. Typically, use a design speed within 30 km/h of the mainline design speed.
2. Lane Balance. Maintain lane balance at the exit and entrance points of the C-D roadways; see Section 12.3.5.
3. Width. C-D roadways may be one or two lanes, depending upon the traffic volumes and weaving conditions. C-D roadways are designed similar to ramps with travelled way widths of either 5.0 m or 7.3 m.
4. Separations. The separation between the C-D roadway and mainline should be as wide as practical. Figure 12-49 provides the minimum separation that should be provided.

Figure 12-49: Separation Between C-D Roadway and Mainline



(a) With Concrete Barrier



(b) With Pier and Concrete Barrier

*Consider providing an additional clearance of 0.6 m. See the Abu Dhabi *Roadside Design Guide* (28).

12.5.3 High-Speed Direct/Semi-Direct Roadways

High-speed direct or semi-direct roadways, like ramps, accommodate turning movements at interchange facilities, but they are distinguished from ramps in that they provide two-lane, one-way operations. These roadways are provided for large traffic movements that exceed the capacity of a one-lane ramp, for route continuity, or for improved traffic operations. For route continuity purposes, direct roadways may carry any route including expressways and freeways. These roadways are directional or semi-directional in alignment and generally have divergence and convergence terminal designs. Design criteria for these roadways are:

1. Direct Roadways. The design speed of direct roadways, in rural areas may be 100 km/h or 120 km/h. In urban areas, direct roadways may be designed for 80 km/h, 90 km/h, or 100 km/h depending on traffic volumes, right-of-way, motorist expectations, and importance of route. In all cases, the maximum superelevation rate is 6% or 4%, based on the maximum rate used on the mainline. Shoulder widths are the same as the mainline roadway.

2. Semi-Direct Roadways. Desirably, use the criteria for direct roadways. However, a minimum design speed of 90 km/h in rural areas and 80 km/h in urban areas may be used.

12.5.4 Design Speed

Table 12-5 provides the ranges of ramp design speeds based on the design speed of the mainline.

Table 12-5: Guide Values for Ramp Design Speed

Mainline Design Speed	80 (km/h)	90 (km/h)	100 (km/h)	120 (km/h)	130 (km/h)	140 (km/h)
	Ramp Design Speed (km/h)					
High Range	70	80	90*	110*	110*	120*
Middle Range	60	60	70	90*	90*	100*
Low Range	40	50	50	70	70	80

* *In general, design speeds greater than 80 km/h apply to two-lane direct or semi-direct roadways within interchanges.*

Source: (1) *Revised*

In addition to Table 12-5, consider the following when selecting the ramp design speed:

1. Loop Ramps. Design speeds in the middle and high ranges are generally not attainable for loop ramps. The following apply to loop ramps:
   - For loop ramps on collector-distributor roadways or in restricted urban conditions, the minimum design speed for loop ramps should be 40 km/h.

- Where the truck ADT is greater than 15%, the design speed for the initial curve after the exit curve shall be determined to facilitate the safety movement of the traffic..

- For rural loop ramps, a 50 km/h design speed is desirable.

- Use a design speed of 50 km/h (maximum) for cloverleaf interchange loop ramps between freeways.

2. Outer Connector Ramps. The design speed for the outer connector ramp of a rural cloverleaf interchange should be 90 km/h or less. Where a wrap-around type ramp is used, use a minimum design speed of 70 km/h for the centre curve.

3. Semi-Direct Connection Ramps. Use design speeds in the middle to high ranges for semi-direct connection ramps. Do not use a design speed less than 60 km/h.

4. Direct Connection Ramps. These include both diagonal ramps at a diamond interchange and ramps at a directional interchange. The design speed should desirably be in the high range.

5. Direct Roadways. Two-lane direct roadways within an interchange are typically designed with a design speed in the high range.

6. Controlled Terminals. If a ramp is terminated at an intersection with a stop or signal control, the design speeds in Table 12-5 are not applicable to a portion of the ramp near the intersection. The design speed on the ramp near the crossroad intersection is usually 60 km/h, but can be a minimum of 40 km/h in constrained areas.

7. Variable Speeds. The ramp design speed may vary based on the two design speeds of the intersecting roadways. Use a higher design speed on the portion of the ramp near the higher speed facility and a lower design speed near the lower speed facility. When using variable design speeds, the maximum speed differential between controlling design elements (e.g. horizontal curves, vertical curves) should not be greater than 20 km/h to 30 km/h. The designer must ensure that sufficient deceleration distance is available between design elements with varying design speeds (e.g. two horizontal curves).

Table 12-6 presents geometric design criteria for interchange ramps based on the selected design speed (e.g. sight distance, horizontal alignment, and vertical alignment). These are discussed in detail in the following sections.

12.5.5 Sight Distance

The designer should review the ramp cross-section, horizontal alignment, and vertical alignment to ensure that stopping sight distance is continuously provided along the interchange ramp. Because ramps are composed of curves of various radii and design speeds, sight distance requirements may vary over the length of the ramp. Table 12-6 provides a summary of the geometric criteria for ramps, including stopping sight distance.

Table 12-6: Summary of Geometric Design Criteria for Interchange Ramps

Geometric Requirements								
Ramp Design Speed (km/h)	**110**	**100**	**90**	**80**	**70**	**60**	**50**	**40**
Stopping Sight Distance (m)	220	185	160	130	105	83	65	50
Horizontal Alignment								
Minimum Radius (m) (e_{max} = 4%)	635	492	375	280	203	135	86	47
Minimum Radius (m) (e_{max} = 6%)	561	437	336	252	184	123	79	43
Minimum Length of arc (m)	See Table 12-8							
Vertical Alignment								
Maximum Upgrades	+3% — +5%					+4% — 6%	+5% — +7%	
Maximum Downgrades	-3% — -5%					-4% — -6%	-5% — - 7%	
Crest Vertical Curves K-Values	74	52	39	26	17	11	7	4
Sag Vertical Curves K-Values	55	45	38	30	23	18	13	9

12.5.6 Cross Section Elements

Figure 12-50 presents the typical cross section criteria for tangent and superelevated portions of ramps. The following also applies to the ramp cross section:

1. Width. Ramp travelled-way widths are governed by the type of operation, curvature, and volume and type of traffic. It should be noted that the roadway width includes the travelled-way width plus the shoulder width or equivalent offset outside the edges of the travelled way. For diagonal ramps, the ramp travelled way width is 5.0 m with a 1.2 m wide left shoulder, and a 3.0 m wide right shoulder with no curb clearance on both sides. Design widths of loop ramp travelled ways for various conditions are provided in Table 12-7. Values are shown for three general design traffic conditions, as follows:

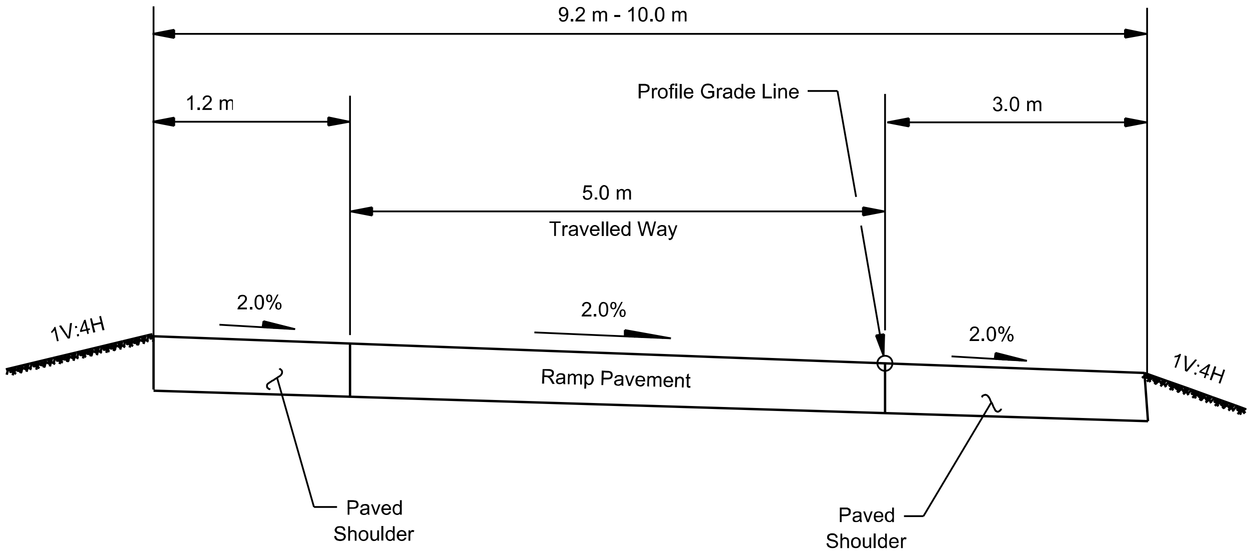
   a. Traffic Condition A. Predominantly P vehicles, but some consideration for SU trucks.

   b. Traffic Condition B. Sufficient SU vehicles to govern design, but some consideration for semitrailer vehicles.

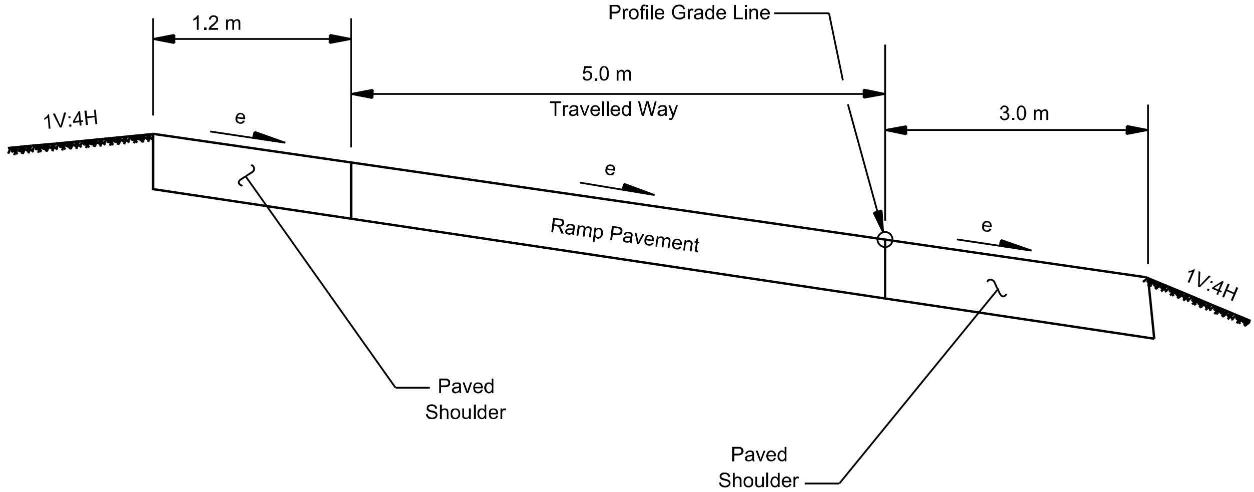
   c. Traffic Condition C. Sufficient buses and combination trucks to govern design.

Traffic conditions A, B, and C are described in broad terms because design traffic volume data for each type of vehicle are not available to define these traffic conditions with precision in relation to travelled-way width. In general, traffic condition A has a small volume of trucks or only an occasional large truck, traffic condition B has a moderate volume of trucks (in the range of 5% to 10% of the total traffic), and traffic condition C has more and larger trucks.

Figure 12-50: Typical Ramp Cross Sections



Tangent Section



Superelevated Section

Table 12-7: Design Widths of Loop Ramps

Radius On Inner Edge of Pavement, R (m)	Pavement Width (m)								
	Case I One-Lane, One-Way Operation – No Provision for Passing Stalled Vehicle			Case II One-Lane, One-Way Operation — With Provision for Passing Stalled Vehicle			Case III Two-Lane Operation — Either One-Way or Two-Way Operation		
	Design Traffic Conditions								
	A	**B**	**C**	**A**	**B**	**C**	**A**	**B**	**C**
15	5.4	5.5	7.0	6.0	7.8	9.2	9.4	11.0	13.6
25	4.8	5.0	5.8	5.6	6.9	7.9	8.6	9.7	11.1
30	4.5	4.9	5.5	5.5	6.7	7.6	8.4	9.4	10.6
50	4.2	4.6	5.0	5.3	6.3	7.0	7.9	8.8	9.5
75	3.9	4.5	4.8	5.2	6.1	6.7	7.7	8.5	8.9
100	3.9	4.5	4.8	5.2	5.9	6.5	7.6	8.3	8.7
125	3.9	4.5	4.8	5.1	5.9	6.4	7.6	8.2	8.5
150	3.65	4.5	4.5	5.1	5.8	6.4	7.5	8.2	8.4
Tangent	3.65	4.2	4.2	5.0	5.5	6.1	7.3	7.9	7.9

Source: (1)

2. Pavement Design. For pavement design information that is also applicable to ramps, see Abu Dhabi *Pavement Design Manual* (55).

3. Cross Slope. For tangent sections, the 5.0 m travelled way is sloped unidirectional at 2.0% towards the right shoulder. Shoulder cross slopes, for both the paved and unpaved portions, are typically 2.0%. For all superelevated ramps, the ramp travelled way and shoulders are sloped as discussed for open-roadways conditions in Section 5.3.

4. Curbs. If curbs are required, place them on the outside edge of the full-width paved shoulders. See Chapter 3 “Cross Section Elements” for information on the use of curbs.

5. Bridges and Underpasses. Carry the full paved width of the ramp, including the paved shoulders over a bridge.

6. Side Slopes/Ditches. For the ramp proper, use a side slope of 1V:4H or flatter. Chapter 3 “Cross Section Elements” and Abu Dhabi *Roadside Design Guide* (28) provide the applicable design information for side slopes and ditches.

7. Clear Zones. Measure the clear zone from the edge of the travelled way on both sides of the ramp using the criteria in the Abu Dhabi *Roadside Design Guide* (28).

8. Right-of-Way. The right-of-way adjacent to the ramp is fully access controlled.

12.5.7 Two-Lane Loop Ramps

The loop ramp is inherently a single-lane facility. However, an individual loop can be adapted into two-lane operation. The following features are essential to achieve effective two-lane operation on loop ramps:

- The two-lane loop configuration should not be immediately preceded or followed by a loop ramp. Appropriate arrangements are loop ramps with directional interchanges with one loop or two loops in opposite alternate quadrants.

- Ramp exits and entrances should comply with the two-lane terminal criteria.

- The continuation of the exit terminal should be direct and gradual, forming a stem road speed transition area 180 m or more in length to align the vehicles in two lanes and then smoothly lead the traffic into the controlling radius of the loop.

- The loop radius of the inner edge of the travelled way normally should be not less than 60 m. A radius of 55 m may be employed under restricted conditions or where the design speed of the approach roadway is 80 km/h or less. A radius of 45 m has been observed to satisfactorily operate with moderate volumes and sufficient width, and where the approach roadway design speed is 70 km/h or less.

- Loop width should provide a travelled way of generally no less than 9.0-m, two 4.5-m lanes). For radii less than 60 m, a width of 9.6 m is appropriate.

12.5.8 Horizontal Alignment

12.5.8.1 Theoretical Basis

Establishing horizontal alignment criteria for any roadway element requires a determination of the theoretical basis for the various alignment factors. These include the side-friction factor (f), the distribution method between side friction and superelevation, the relative longitudinal gradients, and the distribution of the superelevation runoff length between the tangent and horizontal curve. For horizontal alignment on the ramp proper, the theoretical basis will be open-roadway conditions as discussed in Chapter 5 “Horizontal Alignment.” In summary, this includes:

- relatively low side-friction factors (i.e. a relatively small level of driver discomfort);
- the use of AASHTO Method 5 to distribute side friction and superelevation;
- relatively flat longitudinal gradients for superelevation runoff lengths; and
- distributing 67% of the superelevation runoff length on the tangent and the remainder on the horizontal curve.

The following sections discuss the specific horizontal alignment criteria for ramps.

12.5.8.2 Design Controls

The following will apply to the horizontal alignment of ramps:

1. Curve Types. Ramps are commonly designed with simple curves, two-centred compound curves, three centred compound curves, and reverse curves. Figure 12-51 illustrates the use of simple, two-centred, and three-centred compound curves on ramps. For more guidance on curve types and transitions, see Chapter 5 “Horizontal Alignment.”

2. Minimum Curve Radii. Table 12-6 provides the minimum curve radii based on ramp design speed, open-roadway conditions, and e_{max}.

3. Superelevation Rates. The maximum superelevation rate on the ramp may be either 4% or 6% depending upon the design constraints.

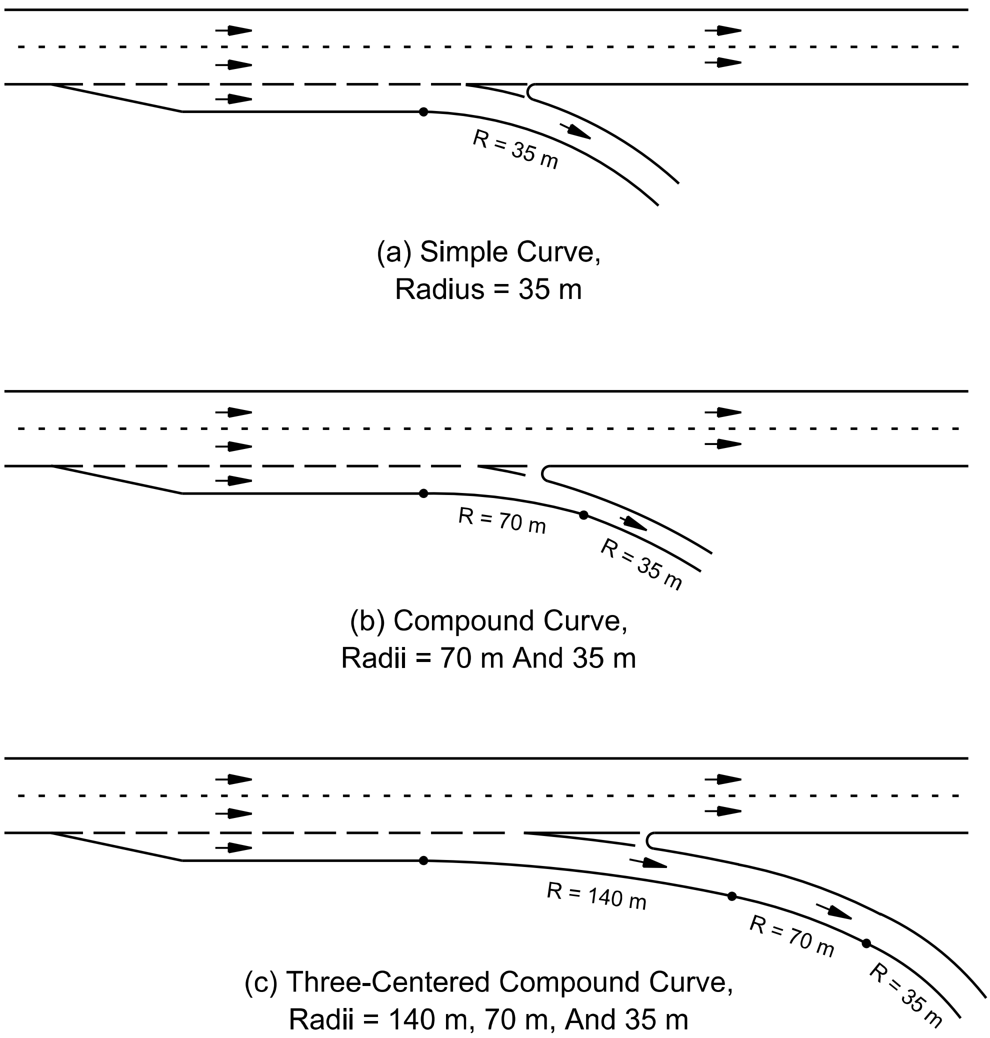
4. Trucks. Where there are a significant number of trucks on loop ramps, the designer may need to consider how the design may increase the rollover potential for large trucks. To reduce this potential, consider using flatter curve radii on the second curve. Modified radii can be obtained by reducing the superelevation rates and/or lowering the side-friction factors.

5. Superelevation Runoff Lengths. Open-roadway conditions, as discussed in Section 5.3, apply to transitioning the ramp from its normal cross slope on tangent to the needed superelevation on curves.

6. Ramp Baseline. Typically, the right edge of the ramp travelled way is used for horizontal and vertical control, and the control point for the axis of rotation.

7. Sight Distance. Section 5.5 presents the criteria for sight distance around horizontal curves based on the curve radii and design speed. These criteria also apply to curves on ramps.

Figure 12-51: Curve Types on Exit Ramps



Source: (1)

8. <u>Reverse Curves</u>. Reverse curves may be required to:

   - meet restrictive right-of-way conditions,
   - provide for a better location of the intersection on the crossroad, and/or
   - provide a preferred angle of intersection with the crossroad.

   Design the reverse curves with a minimum tangent section consisting of a continuously rotating plane between the curves. This continuously rotating plane will determine the necessary distance between the PT and the succeeding PC.

9. <u>Controlled Ramp Termini</u>. Exit ramps may end at a controlled intersection — stop control or signal control. If horizontal curves on the ramps are near the intersection, a design speed for the curve should be selected that is appropriate for expected operations at the curve.

12.5.8.3 Length of Arc

Where compound arcs of decreasing radius are used on exit ramps, the arcs should have sufficient length to enable motorists to decelerate at a reasonable rate over the range of design speeds; see Table 12-8. The radii of the flatter arc compared to the radii of the sharper arc should not exceed a ratio of 2:1 to prevent abruptness in operation and appearance.

Comparable radii and length controls may be used on entrance ramps with compound arcs of increasing radii. However, for entrance ramps, the 2:1 ratio of compound curves is not critical because the vehicle is accelerating into a curve with a larger radius or into a tangent section.

Table 12-8: Arc Lengths for Compound Curves

Radius (m)	30	50	60	75	100	125	150 or more
Minimum (m)	12	15	20	25	30	35	45
Desirable (m)	20	20	30	35	45	55	60

Note: These lengths are applicable where the ramp curve is followed by a curve 1/2 the radius or preceded by a curve of double radius. The lengths apply to each individual curve.

Source: (1)

12.5.9 Vertical Alignment

12.5.9.1 Grades

Table 12-6 provides the limiting gradients for ramps. The selected gradient is dependent upon several factors. These include:

- Where steep grades are required, locate them within the centre portion of the ramp.
- Locate freeway ramp terminals and approach areas near intersections on as flat a grade as practical; see Section 12.6 for freeway ramp terminal grades and Section 12.7 for grades near ramp/crossroad intersections.
- Ramp grades may affect the location of ramp termini. This may be a concern where the ramp intersects the crossroad at an angle of 70 degrees or less. Section 12.7.1 further discusses the location of ramp/crossroad intersections.

12.5.9.2 Vertical Curvature

Design vertical curves on ramps to meet the stopping sight distance criteria based on the ramp design as presented in Chapter 6 “Vertical Alignment.” Table 12-6 provides the K-values for both crest and sag vertical curves. The ramp profile often assumes the shape of the letter S with a sag vertical curve at one end and a crest vertical curve at the other. In addition, design the vertical curvature adjacent to the standard exit and entrance terminals using a design speed of 80 km/h or greater.

12.5.9.3 Cross Sections Between Adjacent Ramps

Where the horizontal alignment of a ramp is designed to be parallel to an adjacent ramp (e.g. partial cloverleaf, cloverleaf, trumpet interchanges), first establish the profile of the loop ramp and then set the profile of the outer ramp to be approximately parallel to the inner-loop ramp profile. This is accomplished by calculating the left-edge elevations of the loop ramp and matching those elevations for the left-edge elevations of the outer ramp. To ensure the median edges between the two ramps are approximately level, develop a typical cross section during the initial design phase.

12.5.10 Roadside Safety

The criteria in the Abu Dhabi *Roadside Design Guide* (28) (e.g. clear zones, barrier warrants, barrier length of need) will apply to the roadside safety design of interchange ramps.

12.5.11 Ramp Capacity

The capacity of the ramp is influenced by the freeway/ramp terminals and the ramp/crossroad intersection. Note that the *Highway Capacity Manual 2010* (17) does not provide a methodology to determine the level of service on the ramp itself. However, the designer needs to check the demand flow rate of the ramp against the capacity of ramp. Ramp capacities, which are based on the ramp free-flow speed, are shown in Table 12-9.

Table 12-9: Capacity of Ramps

Ramp Free-Flow Speed (km/h)	Capacity of Ramp (pc/h)	
	Single-Lane Ramps	Two-Lane Ramps
> 80	2200	4400
> 65 – 80	2100	4200
> 50 – 65	2000	4000
≥ 30 – 50	1900	3800
< 30	1800	3600

Note: Capacity of ramp does not ensure an equal capacity at its freeway junction. Always check the capacity of the freeway/ramp junction and ramp/crossroad intersection.

Source: (17)

Note that two-lane tight loop ramps may cause operational problems and are undesirable. In lieu of designing a loop ramp with design volumes greater than 1500 vehicles per hour, consider providing a direct ramp.

12.5.12 Ramp Metering

Ramp metering consists of traffic signals installed on entrance ramps in advance of the entrance terminal to control the number of vehicles entering the freeway. Signals may be pretimed or traffic actuated to release the entering vehicles individually or in small (usually two-vehicle) platoons.

Ramp metering offers the potential to reduce congestion and its direct effects through the optimal use of freeway capacity. Metering can significantly reduce freeway crash frequencies by reducing stop and go driving behaviour and smoothing the flow of traffic entering freeway facilities. Ramp metering can also improve overall system performance by increasing average freeway throughput and travel speed, and decreasing travel delay. Metering may be limited to only one ramp or integrated into a series of entrance ramps.

Pre-timed metering releases vehicles at regular intervals that have been determined by traffic studies and, usually, simulation modelling. Traffic-actuated metering involves detectors used to measure the traffic conditions on the freeway main line and ramp. The metering rate is determined through one of a number of algorithms. Traffic-actuated metering can be based solely on local conditions on the ramp and on the freeway adjacent to the ramp or on conditions throughout the corridor or freeway system.

Figure 12.52 provides the guidelines for the placement of a signal on the ramp.

12.6 Freeway Ramp Terminals

12.6.1 Exit Ramp Terminals

12.6.1.1 Types

The exit ramp terminal is a speed-change lane that permits high-speed traffic to exit from the through lane of the roadway and enter the ramp proper. The exit terminal must be visible to the approaching motorist and provide a clear indication for the point of departure from the travelled way. When designing the interchange, consider the following exit ramp types:

1. Standard Exit Terminal. There are two basic types of exit freeway ramp terminals — the parallel design and the taper design; see Figure 12-53. The Abu Dhabi *Standard Drawings for Road Projects* (27) provide the standard exit ramp terminal design.

2. Exit Terminal with an Auxiliary Lane. An auxiliary lane may be required prior to the exit terminal:

   - to meet the guidelines discussed in Section 12.3.7;

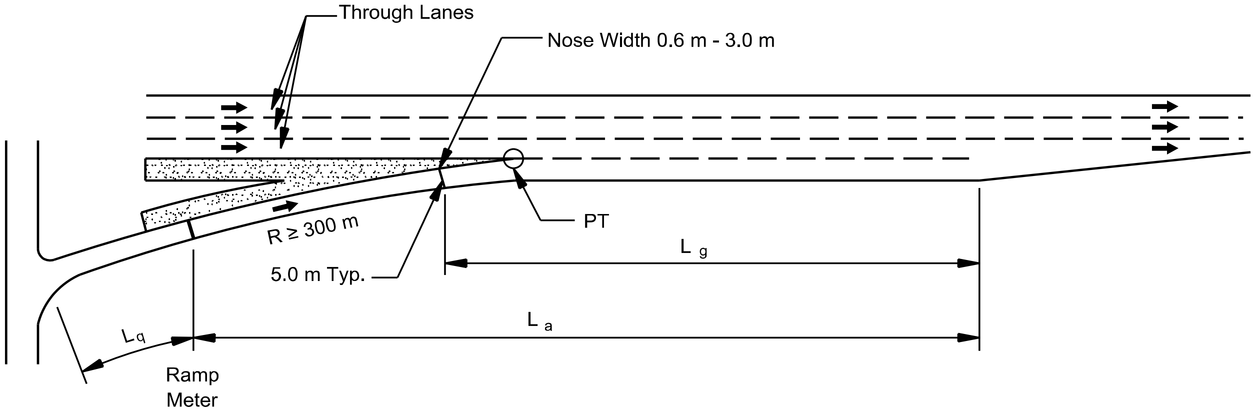
   - where the exiting design traffic exceeds the appropriate service volume of a standard exit terminal design, but does not require a two-lane exit; and/or

   - proceeding a left-hand exit terminal. Note that interchange designs should not use left-hand exit terminals. Figure 12-53(b) illustrates the design criteria for an exit terminal preceded by an auxiliary lane. Provide transverse pavement markings in the recovery area to discourage the use of the auxiliary lane beyond the exit gore. Pavement markings in the recovery area should be according to the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

3. Two-Lane Exits. These terminals are typically required where the traffic volumes on the ramp exceed the capacity of a single-lane exit ramp. The designer should consider the following for two-lane exit terminals:

   a. Lane Balance. For consistent freeway operations, maintain lane balance at the freeway ramp terminal; see Section 12.3.5.

b. Design. A typical two-lane exit terminal design is illustrated in Figure 12-54. To develop the proper level of service at a two-lane exit facility and provide proper lane balance, add a minimum 450 m long auxiliary lane prior to the exit terminal.



(a) Parallel Design

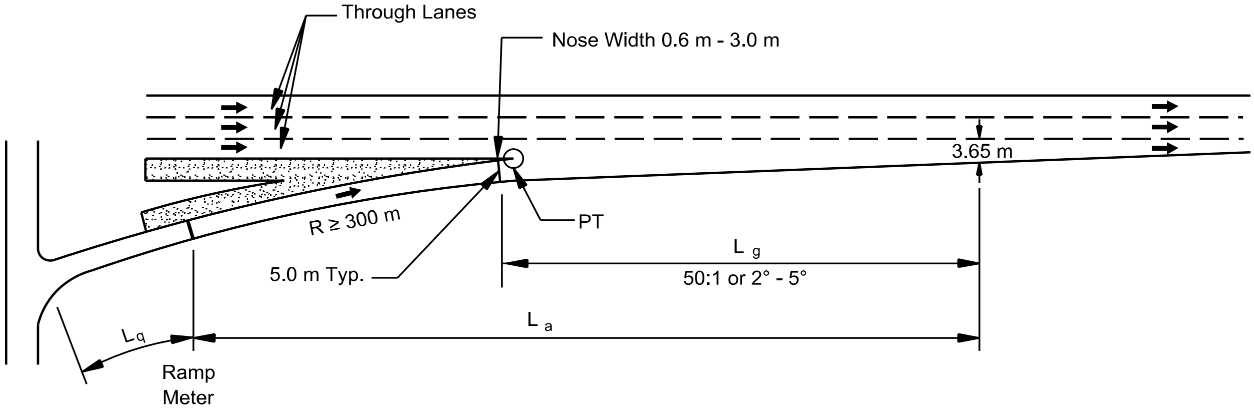


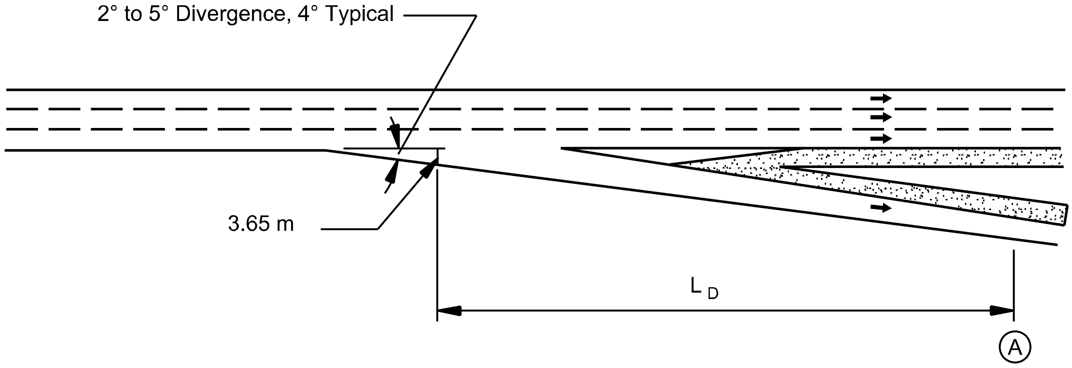
Figure 12-52: Ramp Metering Signal Location

(b) Tapered Design

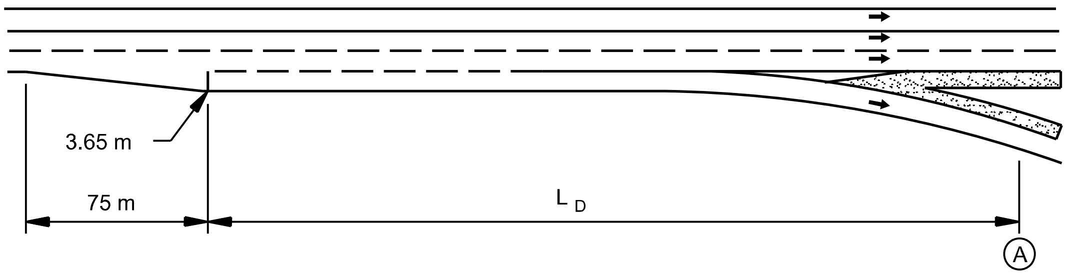
Notes:

1. *L_a is the required acceleration length as shown in Table 12-13 or as adjusted by Table 12-14.*
2. *L_a should not start back on the curvature of the ramp unless the radius equals 300 m or more.*
3. *L_g is the required gap acceptance length. L_g should be a minimum of 150 m, depending on the nose width.*
4. *L_q is the necessary distance for backup queue from the ramp meter to the intersection. Ensure the queue does not backup into the intersection.*
5. *The value of L_a or L_g, whichever produces the greater distance downstream from where the nose equals 0.6 m is suggested for use in the design of the ramp distance.*

Figure 12-53: Exit Ramp Terminals



(a) Tapered Design



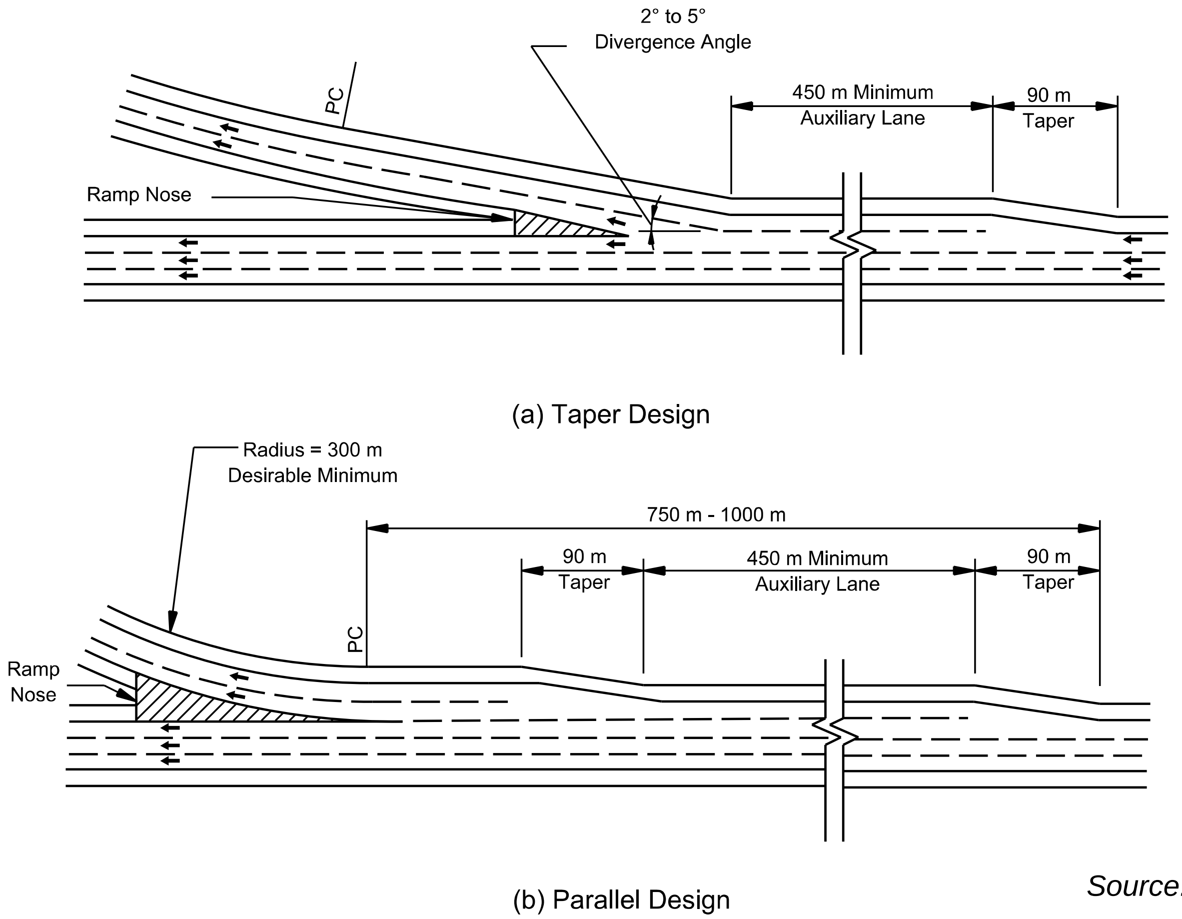
(b) Parallel Design

Notes:

1. *L_D as shown in Table* 12-10.
2. *Point A controls the speed of the ramp, typically the horizontal curvature.*

Source: (1)

Figure 12-54: Two-Lane Exit Ramp Terminals



Source: (1)

12.6.1.2 Length

Table 12-10 provides the deceleration lengths based on design speed. Consider the following when determining the appropriate length of an exit ramp terminal:

1. Capacity. Where the mainline and ramp will carry traffic volumes approaching the design capacity of the merge area, consider using an auxiliary lane exit ramp design as discussed in Section 12.6.1.1.

2. Trucks. Research has shown the deceleration distances provided in Table 12-10 are also reasonable for trucks (56).

3. Gradients. Where the gradient of the mainline and/or ramp exceed -3%, the deceleration length may need to be adjusted. These adjustments are shown in Table 12-11. For upgrades, use the standard exit terminal design, and do not reduce the deceleration distance.

Table 12-10: Design Lengths for Deceleration Lanes (Passenger Cars)

Design Speed of Roadway (km/h)	Speed Reached at End of Full Lane Width (km/h) (V_a)②	L = Length of Deceleration Lane Excluding Taper (m)① For Design Speed of Turning Roadway (km/h)											
		Stop	20	30	40	50	60	70	80	90	100	110	120
		For Average Running Speed of Exit Curve (km/h) (V'_a)											
		0	20	28	35	42	51	63	70	77	85	91	98
50	47	75	70	60	45	—	—	—	—	—	—	—	—
60	55	95	90	80	65	55	—	—	—	—	—	—	—
70	63	110	105	95	85	70	55	—	—	—	—	—	—
80	70	130	125	115	100	90	80	55	—	—	—	—	—
90	77	145	140	135	120	110	100	75	60	—	—	—	—
100	85	170	165	155	145	135	120	100	85	55	—	—	—
110	91	180	180	170	160	150	140	120	105	80	55	—	—
120	98	200	195	185	175	170	155	140	120	105	80	60	—
130	102	225	220	210	205	195	180	155	135	115	95	75	—
140	110	250	240	235	225	220	205	180	165	145	120	105	80

Notes:

1. *These values are for grades 3% or less. See Table* 12-11 *for steeper downgrades.*
2. *The deceleration lengths are calculated from the distance needed for a passenger car to decelerate from the average running speed of the mainline to reach a speed (V'_a) of the average running speed on the exit ramp.*

Source: (1) (57)

Table 12-11: Grade Adjustments for Deceleration (Passenger Cars)

Design Speed (km/h)	Ratio of Length on Grade to Length on Level for Design Speed of Ramp (km/h) ①					
	40	50	60	70	80	All Speeds
All speeds	3% to 4% Upgrade 0.9					3% to 4% Downgrade 1.2
All speeds	5% to 6% Upgrade 0.8					5% to 6% Downgrade 1.35

Notes:

1. *Where a deceleration lane is proposed on a grade greater than -3%, select a length of lane from Table* 12-10 *and multiply that value by the ratio obtained from above to determine the design length on grade.*
2. *The grade in the table is the average grade measured over the distance for which the deceleration length applies.*

Source: (1)

4. Horizontal Curves. The application of the deceleration criteria regarding horizontal curves preceding the exit terminal are as follows:

a. Design Speed. The design speed of the horizontal curve adjacent to an exit terminal should be determined by open-roadway conditions.

b. Curve Radii/Tangent Lengths. Provide the minimum controlling ramp curve radii and the minimum tangent lengths that should be used prior to the standard exit terminal based on the mainline design speed.

5. Additional Lengths. If it has been determined that additional deceleration distance is required beyond that provided with the standard exit terminal, the following options may be considered:

a. Typical Design. If substantial additional distance is required for deceleration, use the auxiliary lane terminal design as shown in Figure 12-53(b).

b. Optional Design. If only a small additional distance is required to meet the necessary deceleration length, the additional distance may be gained by extending the LD distance. The designer must ensure that this will not create other undesirable aspects in the design of the ramp proper.

c. Low-Volume Conditions. Where existing volumes on the mainline are low and where the slower exiting vehicles will not reduce the level of service on the mainline, the use of the standard exit terminal may be considered. However, provide sufficient right-of-way so that an auxiliary lane can be added in the future.

d. Secondary Impacts. Before providing any additional deceleration lane length, the designer must consider its impacts (e.g. additional construction costs, wider structures, right-of-way impacts).

12.6.1.3 Sight Distance

The sight distance approaching the gore nose should exceed the stopping sight distance (Section 4.3) for the through traffic, desirably by 25% or more. Where there are unusual conditions, consider providing decision sight distance to the exit terminal (Section 4.4). Extra sight distance is particularly important for exit loops immediately beyond a structure. When measuring for adequate sight distance, desirably the motorist should be able see the pavement surface at and beyond the gore nose. Locating the exit terminal and gore nose where the mainline is on an upgrade provides the best design condition. Do not locate exit terminals near mainline crest vertical curves where the ramp pavement may disappear from the driver’s view.

12.6.1.4 Alignment

The preferred practice is to locate exit terminals on tangent sections or on mainline curves to the right. However, this may not be practical in highly restricted areas. Section 12.5.7 discusses the minimum alignment criteria for the ramp proper, including the minimum radii for the initial ramp curve (R).

12.6.1.5 Superelevation and Cross Slopes

Ramp cross slopes and superelevation rates for horizontal curves on ramps near the freeway ramp terminal must be developed to properly transition the driver from the mainline to the first curve on the exit ramp. The following will apply:

1. Cross Slope. The cross slope of the initial segment of the ramp departure from the through lane, or an auxiliary lane preceding the exit ramp is usually sloped at the same rate as the mainline. However, if the mainline has a flat longitudinal grade (i.e. less than 0.35%); consider increasing the cross slope rate on an auxiliary lane and the exit terminal to 2.5%. Where the mainline is curving to the right, the maximum cross slope on the exit terminal is 5%. Where the mainline is curving to the left, the maximum cross slope on the terminal is 4%.

2. Maximum Ramp Superelevation. Use an e_{max} of 4% or 6%.

3. Radius/Superelevation Rate. Section 5.3 discusses the use of Method 5 for open roadway conditions to distribute superelevation and side friction. This theoretical basis also applies to the ramp portion of freeway exit terminals. See Table 12-6 to determine the proper radius and superelevation rate for horizontal curves on exit ramps. To determine the applicable design speed to use, see Table 12-12.

Table 12-12: Minimum Design Speed for Initial Exit Curve

Mainline Design Speed (km/h)	Minimum Design Speed (km/h) of Initial Curve (R_1)
140	90
120	80
100	70
80	60
60*	40

** C-D roads only*

12.6.1.6 Gore Area

The gore area is normally considered both the paved triangular area between the through lane and the exit ramp, plus the graded area that extends a significant distance downstream beyond the gore nose. The following definitions will apply:

1. Physical Nose. This is a point where the 5-m ramp width begins.

2. Gore Nose. This is the point where the paved shoulders separate from each other and the non-paved area begins as the ramp and mainline diverge.

Consider the following when designing the gore area:

1. Roadside Obstacles. Desirably, the area beyond the gore nose should be free of all obstacles (except the ramp exit sign) for at least 30 m or more beyond the gore nose. Any obstacles within approximately 100 m of the gore nose should be made breakaway or shielded by a barrier. See Abu Dhabi *Roadside Design Guide* (28) for additional guidance for the treatment of roadside obstacles.

2. Curbing. Do not use curbing within the gore area of an exit terminal.

3. Side Slopes. Side slopes and ditches adjacent to the gore area should meet the same criteria as the mainline. The graded area beyond the gore nose should be as flat as

practical, but still drain properly. The exit terminal should be located so there are no major elevation differences in this area. For some reconstruction projects, the vertical divergence of the ramp and mainline profiles may warrant protection for both roadways beyond the gore nose (e.g. guardrail and/or impact attenuators).

4. Cross Slopes. The paved triangular gore area between the through lanes and exit ramp should be flat and traversable. The cross slopes in the gore area from the physical nose to the gore nose are 2.0%. This design provides a drainage swale in the neutral area of the terminal.

5. Traffic Control Devices. Signing in advance of the exit and at the divergence should be according to the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

12.6.1.7 Structures

Exit ramp terminals on or near structures can create a split-bridge design which, because of safety, economic, and maintenance considerations, should be avoided. This results in a fixed object on the structure that must be shielded by an impact attenuator. For information on the required minimum impact attenuator area, see Abu Dhabi *Roadside Design Guide* (28).

12.6.2 Entrance Ramp Terminals

12.6.2.1 Types

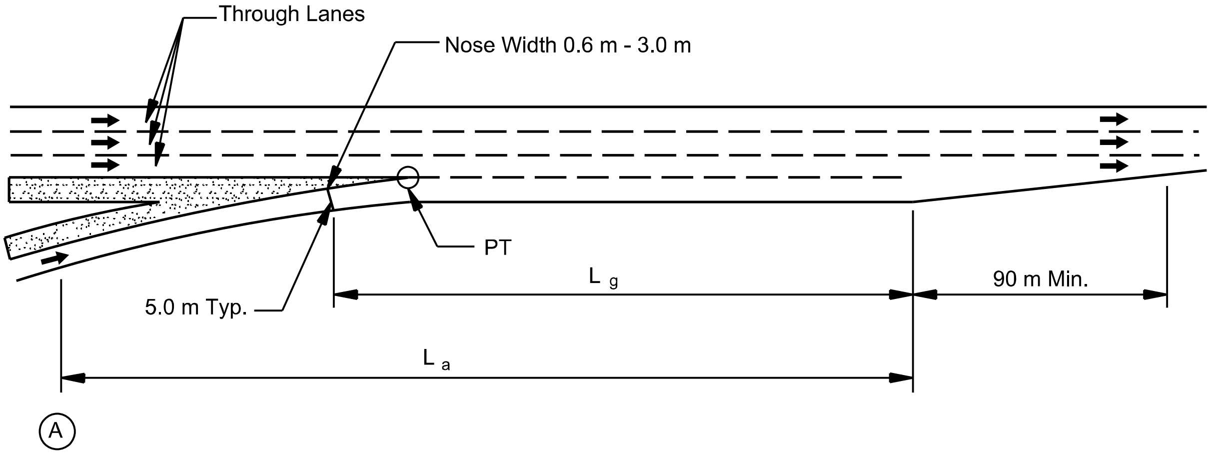
The entrance ramp terminal is a speed-change lane that permits ramp traffic to accelerate and merge with the high-speed traffic on the mainline. When designing the interchange, consider the following entrance ramp types:

1. Standard Entrance Terminal. There are two basic types of entrance roadway ramp terminals — the parallel design and the taper design; see Figure 12-55. In Abu Dhabi, only the parallel design is used. The Abu Dhabi *Standard Drawings for Road Projects* (27) provides the standard entrance ramp terminal designs. Use this ramp design for all single-lane entrances where the level of service of the ramp terminal is equal to or greater than that of the mainline. Where an expressway or freeway merges at an interchange, use the convergence design as discussed in Section 12.8.2.

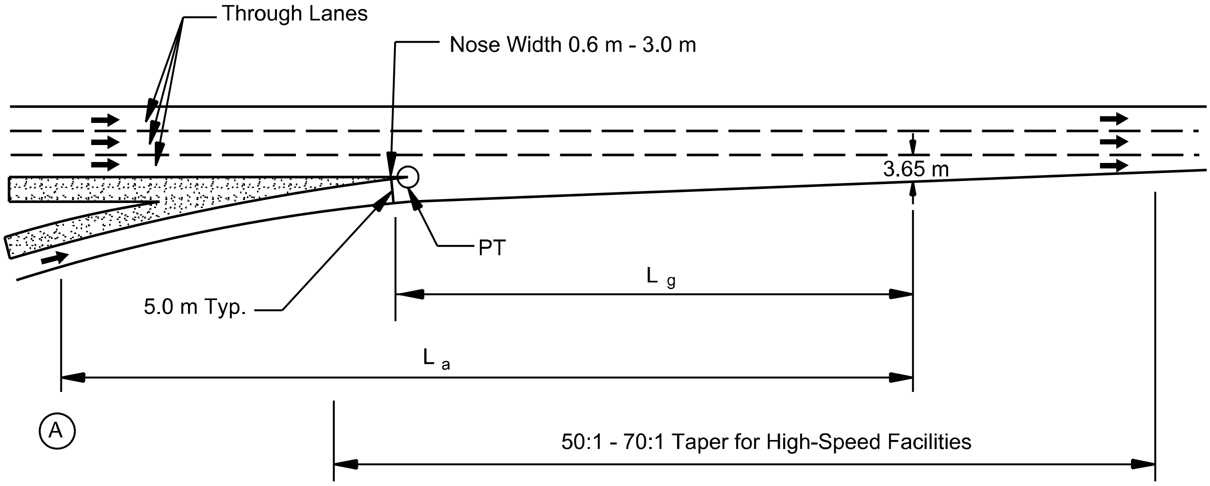
2. Entrance Terminal with an Auxiliary Lane. An auxiliary lane may be required after the entrance terminal:

- to meet the requirements in Section 12.3.7, and/or
- where the entering traffic exceeds the appropriate service volume of a standard entrance terminal design but where a two-lane entrance ramp is not required.

Figure 12-55: Entrance Ramp Terminals



(a) Parallel Design



(b) Tapered Design

Notes:

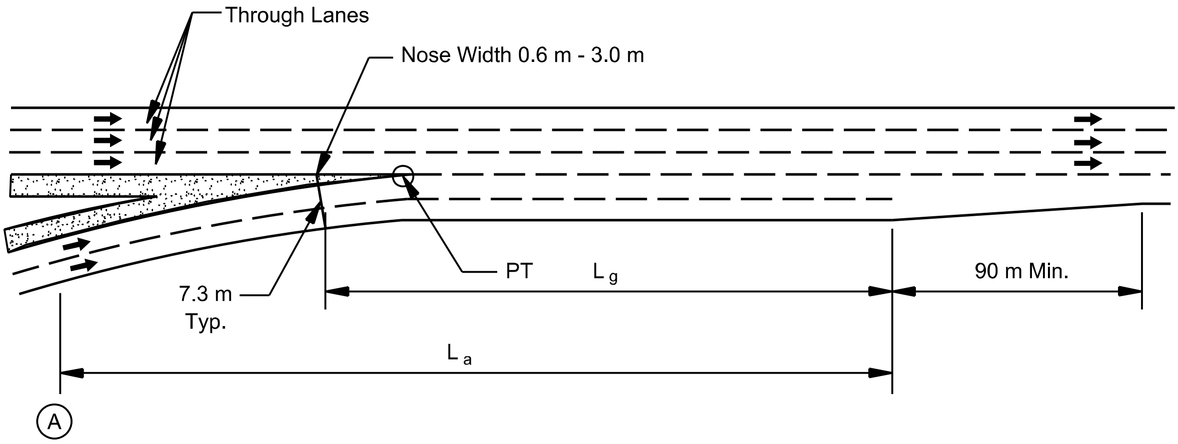
1. *L_a is the required acceleration length as shown in Table 12-13 or as adjusted by Table 12-14.*
2. *Point A controls speed on the ramp. L_a should not start back on the curvature of the ramp unless the radius equals 300 m or more.*
3. *L_g is the required gap acceptance length. L_g should be a minimum of 150 m, depending on the nose width.*
4. *The value of L_a or L_g, whichever produces the greater distance downstream from where the nose equals 0.6 m is suggested for use in the design of the ramp distance.*

Figure 12-55 illustrates the design criteria for an entrance terminal with an auxiliary lane. The final ramp radius typically is 252 m (e_{max} = 6%) or 280 m (e_{max} = 4%). Typically, the auxiliary lane should be at least 300 m. Where the final ramp radius is less than 252 m (280 m), the length of the auxiliary lane will be based on the necessary acceleration distance as discussed in Section 12.6.2.2.

3. Two-Lane Entrances. Where the entrance design traffic exceeds the service volume of a single-lane entrance ramp terminal with an auxiliary lane, it may be necessary to provide a two-lane entrance terminal as illustrated in Figure 12-56. In Abu Dhabi, only the parallel design is used. Where a two-lane entrance ramp is required, an additional lane on the freeway is necessary to accommodate the additional traffic. This lane may be dropped 600 m to 900 m downstream or at the next interchange.

If the two-lane entrance is preceded by a two-lane exit ramp terminal, an increase in the basic number of lanes will generally not be required. In this case, the added lane that results from the two-lane entrance is considered an auxiliary lane. Note that this design violates lane balance guidelines discussed in Section 12.3.5. Where the demand volume of the entering traffic exceeds this design or where the entering roadway is a freeway or expressway, use the major convergence design as discussed in Section 12.8.2.

Figure 12-56: Two-Lane Entrance Ramp Terminal



12.6.2.2 Length

Consider the following when determining the appropriate length of an entrance terminal:

Through lanes

1. Lengths. Right-turn acceleration lanes should meet the criteria presented in Table 12-13. The controlling curve is the design speed of the ramp at first horizontal curve.

2. Gradients. Where the gradient of the mainline and/or ramp exceed +3%, the acceleration length may need to be adjusted. These adjustments are shown in Table 12-14. For downgrades, use the standard entrance terminal design, and do not reduce the acceleration distance.

3. Capacity. Where the mainline and ramp will carry traffic volumes approaching the design capacity of the merge area, consider using an auxiliary lane entrance ramp design as discussed in Section 12.6.3.

4. Trucks. Where there are a significant number of trucks to impact the level of service on the freeway and ramp, acceleration lanes may need to be treated as truck-climbing lanes. Figure 6-7 provides truck acceleration distances. Typical areas where trucks might govern the ramp design include weigh stations, rest areas, truck stops, and transfer staging terminals. In addition, consider using truck acceleration criteria where there is substantial entering truck traffic and the interchange crossroad has a high-skew angle or there is a significant crash history involving trucks attributable to an inadequate acceleration length.

5. Horizontal Curves. The application of the acceleration criteria regarding horizontal curves preceding the entrance terminal are as follows:

   a. Design Speed. The design speed of the horizontal curve adjacent to an entrance terminal should be determined by open-roadway conditions.

   b. Curve Radii/Tangent Lengths. Provide the minimum controlling ramp curve radii and the minimum tangent lengths that should be used prior to the standard entrance terminal based on the mainline design speed.

6. Additional Lengths. If it has been determined that additional acceleration distance is required beyond that provided with the standard entrance terminal, the following options may be considered:

   a. Low-Volume Conditions. Where existing volumes on the mainline are low and where the slower entering vehicles will not reduce the level of service on the mainline, the use of the standard entrance terminal may be considered. The speed profile of merging trucks onto the mainline must be investigated. However, provide sufficient right-of-way so that an auxiliary lane can be added in the future.

   b. Secondary Impacts. Before providing any additional acceleration lane length, the designer must consider its impacts (e.g. additional construction costs, wider structures, right-of-way impacts).

7. Taper. The taper at the end of the parallel ramp terminal should be a minimum of 90 m.

Table 12-13: Design Lengths for Acceleration Lanes (Passenger Cars)

Design Speed of Freeway (km/h)	Speed Reached at End of Full Lane Width (km/h) (V_a)②	L = Length of Acceleration Lane Excluding Taper (m) ① For Design Speed of Ramp (km/h)											
		Stop	20	30	40	50	60	70	80	90	100	110	120
		For Average Running Speed (km/h) (V'_a)											
		0	20	28	35	42	51	63	70	77	85	91	98
50	37	60	50	30	—	—	—	—	—	—	—	—	—
60	45	95	80	65	45	—	—	—	—	—	—	—	—
70	53	150	13	110	90	65		—	—	—	—	—	—
80	60	200	18	165	145	115	65	—	—	—	—	—	—
90	67	260	24	225	205	175	125	35	—	—	—	—	—
100	74	345	32	305	285	255	205	110	40	—	—	—	—
110	81	430	41	390	370	340	290	200	125	50	—	—	—
120	88	545	53	515	490	460	410	325	245	155	45	—	—
130	92	595	58	560	540	510	455	380	305	220	110	—	—
140	100	705	69	675	655	625	575	510	440	350	245	155	40

Notes:

1. *These values are for grades 3% or less. See Table* 12-14 *for steeper upgrades or downgrades.*
2. *The acceleration lengths are calculated from the distance needed for a passenger car to accelerate from the average running speed of the entrance curve to reach a speed (V_a) of approximately 8 km/h below the average running speed on the mainline.*

Source: (1) (57)

Table 12-14: Grade Adjustments for Acceleration (passenger cars)

Design Speed of Roadway (km/h)	Difference of Length on Grade to Length on Level Design Speed of Acceleration Lane (km/h)					
	40	50	60	70	80	All speeds
		3% to 4% Upgrade				3% to 4% Downgrade
60	1.3	1.4	1.4	—	—	0.7
70	1.3	1.4	1.4	1.5	—	0.65
80	1.4	1.5	1.5	1.5	1.6	0.65
90	1.4	1.5	1.5	1.5	1.6	0.6
100 – 140	1.5	1.6	1.7	1.7	1.8	0.6
	5% to 6% Upgrade					5% to 6% Downgrade
60	1.5	1.5	—	—	—	0.6
70	1.5	1.6	1.7	—	—	0.6
80	1.5	1.7	1.9	1.8	—	0.55
90	1.6	1.8	2.0	2.1	2.2	0.55
100	1.7	1.9	2.2	2.4	2.5	0.5
110	2.0	2.2	2.6	2.8	3.0	0.5
120	2.3	2.5	3.0	3.2	3.5	0.5
130	2.6	2.8	3.4	3.6	4.0	0.5
140	2.9	3.1	3.8	4.0	4.4	0.5

Notes:

1. *Where an acceleration lane is proposed on a grade greater than 3%, select a length of lane from Table* 12-13 *and multiply that value by the ratio obtained from above to determine the design length on grade.*
2. *No adjustment is needed on grades 3% or less.*
3. *The grade in the table is the average grade measured over the distance for which the acceleration length applies.*

Source: (1) (57)

12.6.2.3 Sight Distance

Decision sight distance desirably should be provided for drivers on the mainline approaching an entrance terminal. They need sufficient distance to see the merging traffic and adjust their speed or change lanes to allow the merging traffic to enter the freeway. Likewise, drivers on the entrance ramp need to see a sufficient distance to locate gaps in the traffic stream for merging. Section 4.4 provides decision sight distances.

12.6.2.4 Superelevation and Cross Slopes

Standard entrance terminal designs are provided in the Abu Dhabi *Standard Drawings for Road Projects* (27).

12.6.2.5 Gore Area

The following presents the nose criteria for entrance gores:

1. Physical Nose. This is where the edge of travelled way for the freeway and edge of travelled way for ramp theoretically meet.

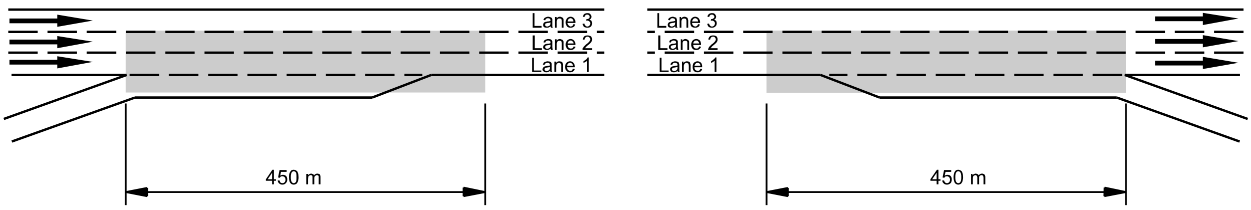
2. Gore Nose. This is a point where the paved shoulders are 600 mm apart.

12.6.3 Capacity Analysis

Ramp-freeway junctions create turbulence in the merging or diverging traffic stream. In general, the turbulence is the result of high lane changing. This turbulence affects both the traffic on freeway and the ramp. The *Highway Capacity Manual 2010* (17) provides the methodology that should be used determine the capacity of ramp merges and diverges. The designer should consider the following:

1. Influence Area. Figure 12-57 illustrates the influence areas for merges and diverges. Note that the influence area also includes lanes 1 and 2 of the freeway mainline.

Figure 12-57: Merge and Diverge Influence Areas



(a) Merge Influence Area (b) Diverge Influence Area

Source: (17)

2. Required Freeway Data. The following information concerning the freeway mainline is required to conduct the analysis:

   - freeway free flow speed;
   - number of travel lanes;
   - terrain (i.e. level, rolling, or mountainous);
   - percentage of heavy vehicles (e.g. trucks, buses);
   - demand flow rate upstream of the freeway-ramp terminal;
   - peak-hour factor; and

- driver population factor.

3. Required Ramp Terminal Data. The following information concerning the ramp terminal is required to conduct the analysis:

    - the type of freeway/ramp terminal (e.g. on-ramp, off-ramp, major merge, major diverge);
    - side of junction (i.e. right side or left side);
    - number of lanes on the ramp;
    - number of lanes in freeway/ramp terminal;
    - length of acceleration/deceleration lane;
    - free flow speed of ramp;
    - terrain (i.e. level, rolling or mountainous);
    - percentage of heavy vehicles (e.g. trucks, buses);
    - demand flow rate on the ramp;
    - peak-hour factor;
    - driver population factor; and
    - location, demand flow rate, peak hour factor, and heavy vehicle percentage of upstream or downstream ramps.

4. Capacity of Ramp-Freeway Terminal. Maximum desirable flow rate entering the merge influence area is 4600 pc/h and for entering the diverge influence area is 4400 pc/h. Note flow rates greater than noted above do not automatically result in a LOS of F, but do require additional interpretation of the results.

5. Level of Service (LOS). LOS for freeway/ramp terminals are defined in terms of density and are shown in Table 12-15.

Table 12-15: Level of Service (LOS) Definitions for Merge and Diverge Segments

Level of Service	Measures of Performance	
	Density (pc/km/ln*)	Number of Stops Before Clearing
A	0 – 6	—
B	6 – 12	—
C	12 – 17	—
D	17 – 22	1 – 2
E	> 22	3 – 4
F	Demand exceeds capacity	

**pc/km/ln – passenger cars per kilometre per lane*

Source: (18)

12.6.4 Branch Connections

A branch connection should be provided when the design year volume exceeds 1500 equivalent passenger cars per hour.

Merging branch connections should be designed as shown in Figure 12-58. Diverging branch connections should be designed as shown in Figure 12-59. The standard ramp exit connects to a local street. The diverging branch connection connects to another primary roadway and has a flatter angle that allows a higher departure speed.

At a branch merge, an 800m length of auxiliary lane should be provided beyond the merge of one lane of the inlet, except where it does not appear that capacity on the primary roadway will be reached until five or more years after the 20 year design period. In this case the length of auxiliary lane should be a minimum of 300m. For diverging connections where less than capacity conditions beyond the design year are anticipated, the length of auxiliary lane in advance of the exit should be 400m.

12.6.4.1 Branch Lane Drops

The lane drop taper on a freeway-to-freeway connector shall not be less than 70:1.

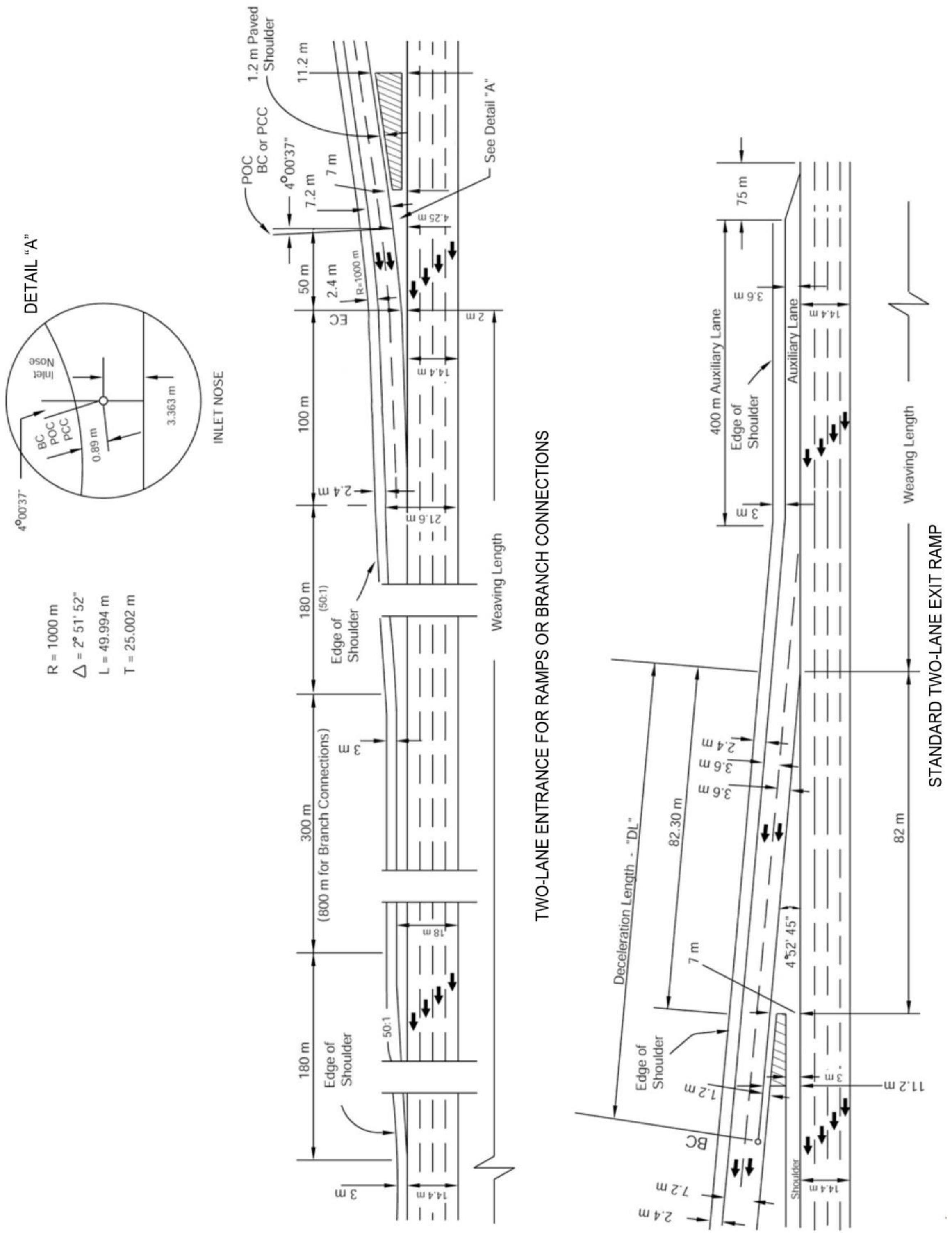


Figure 12-58: Two-Lane Entrance and Exit Ramps

from Caltrans *"Highway Design Manual"*, 2006

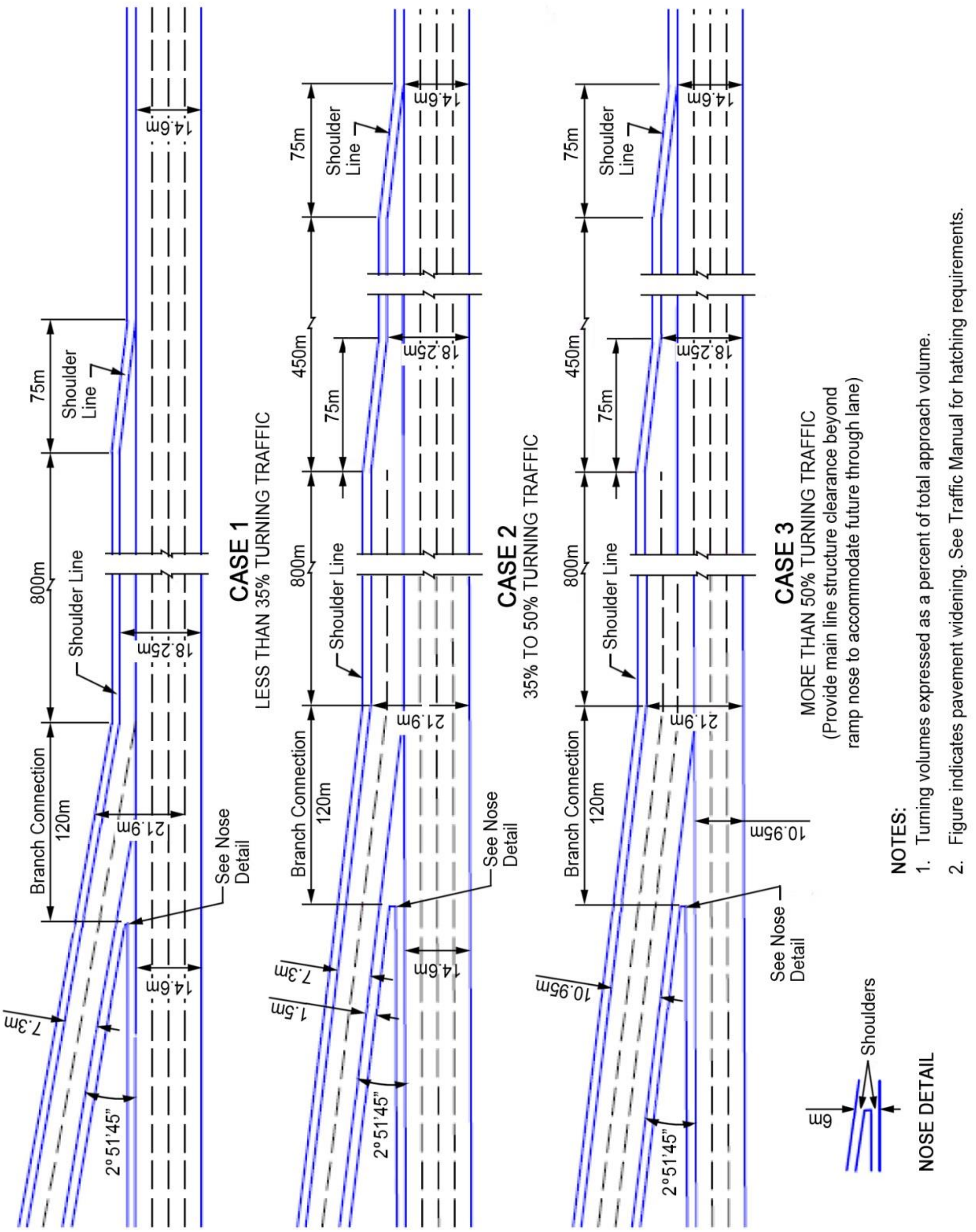


Figure 12-59: Diverging Branch Connections

from Caltrans *"Highway Design Manual"*, 2006

12.7 Ramp/Crossroad Intersections

12.7.1 General Design Criteria

At diamond and partial cloverleaf interchanges, the ramp will terminate or begin at a controlled intersection on the crossroad, with a stop sign, traffic signal, or roundabout. In general, the intersection should be designed as described in Chapter 10 “Intersections” or Chapter 11 “Roundabouts.” Consider the following in the design of ramp/crossroad intersections:

1. Length of Left-Turn Lanes. For diamond interchanges, the minimum distance between ramp/crossroad intersections should be set by the length of overlapping left-turn lanes. Left-turn lanes are usually designed with straight-line tapers when the crossroad goes over the freeway and with reverse curves when the crossroad goes underneath the freeway. For compressed diamond interchanges, the length of the left-turn lanes will be determined based on left-turn storage requirements, see Section 10.7. The left-turn control radii into the ramps are set at the ends of the left-turn lanes. This also determines the location of the ramp baselines.

2. Turn Lanes on Ramps. Exclusive turn lanes are often required at the end of an exit ramp. Chapter 10 “Intersections” provides information on the design of turn lanes at intersections, which are also applicable for ramps.

3. Signalization. Where queuing at one intersection is long enough to effect operations at another, the two intersections may need a larger separation, interconnected signals, or a six-phase overlap signal design.

4. Ramp Grades. Where the exit and entrance ramps intersect with the crossroad, design the first 45 m to 60 m of the ramp with a profile grade of 1.5% to 2%.

5. Crossroad Grades. Design the crossroad grades for a maximum of 2% through the ramp/crossroad intersection.

6. Capacity. Ensure that sufficient capacity and storage for the ramp/crossroad intersection is available. This may require adding lanes at the intersection or on the ramp proper. The analysis must also consider the operational impacts of the traffic characteristics in either direction on the crossroad. For guidance on intersection capacity analysis, see Chapter 10 “Intersections” and the *Highway Capacity Manual 2010* (17).

7. Sight Distance. Section 4.6 discusses the criteria for intersection sight distance. These criteria also apply to the ramp/crossroad intersection. Give special attention to the location of the bridge piers, abutments, sidewalks, bridge railing, roadside barrier, etc.; these elements may present major sight distance obstacles. The bridge obstruction and the required intersection sight distance may result in the relocation of the ramp/crossroad intersection further from the structure. In addition, the crest vertical curve on the crossroad may need to be lengthened to provide adequate sight distance in the vertical plane.

8. Wrong-Way Movements. Wrong-way movements may originate at the ramp/crossroad intersection onto an exit ramp. Figure 12-60 and Figure 12-61 illustrate ramp/crossroad designs that discourage wrong-way entry onto the ramp. It is important to provide signing on

the crossroad and ramp according to criteria in the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

9. Design Vehicle. Design the radius returns and left-turn control radii for ramp/crossroad intersections using a WB-20 design vehicle; see Section 10.2.8. Where applicable, WB-33D design vehicle should physically be able to make the turn.

12.7.2 Slip Ramp Designs

The designer must consider the impact of frontage roads, where present, on interchange design. At some urban interchanges, separating the intersection of the ramp and frontage road with the crossroad may be impractical. In these cases, the only alternative is to provide a slip ramp to a one-way frontage road before the intersection with the crossroad. This can apply to either an exit or entrance ramp. Sufficient distance must then be provided between the freeway ramp terminal and the ramp/frontage road terminal to provide the necessary acceleration or deceleration distance and weaving distance.

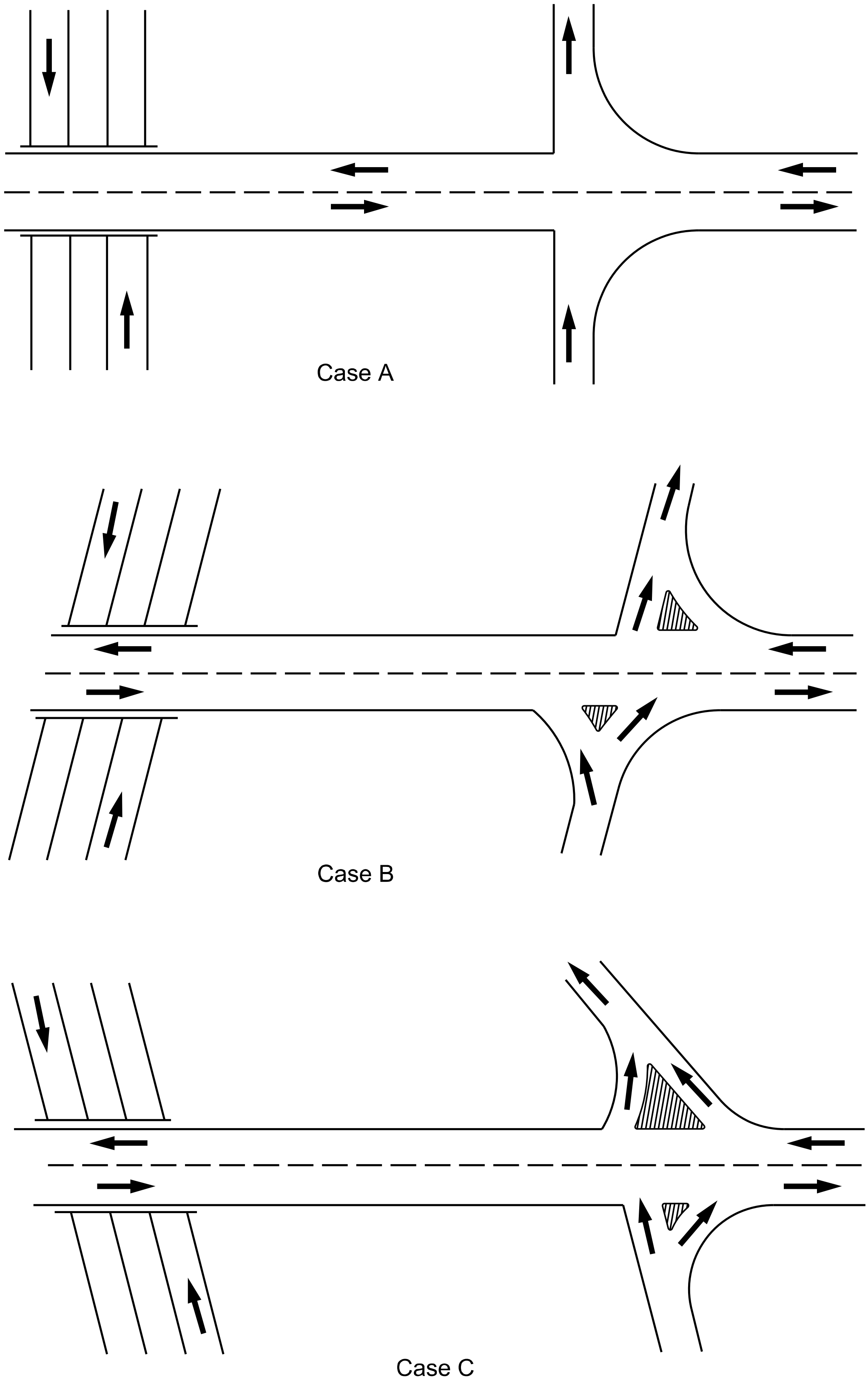
Table 12-16 provides the basic schematic for this design. The critical design element is the distance "A" between the ramp/frontage road merge and the crossroad. This distance must be sufficient to allow traffic weaving, vehicular deceleration and stopping, and vehicular storage to avoid interference with the merge point. Table 12-16 presents general guidelines that may be used to estimate this distance during the preliminary design phase. A number of assumptions have been made including weaving volume, operating speeds, and intersection queue distance. Therefore, a detailed capacity analysis will be necessary to firmly establish the needed distance to properly accommodate vehicular operations.

Distance "B" in Table 12-16 is determined on a case-by-case basis. It should be determined based on the number of frontage road lanes and the intersection design. This distance is typically determined by the necessary weave distance from the intersection to the ramp entrance. For capacity analysis of the weave section, see the *Highway Capacity Manual 2010* (17).

12.7.3 Crossroad Access Control

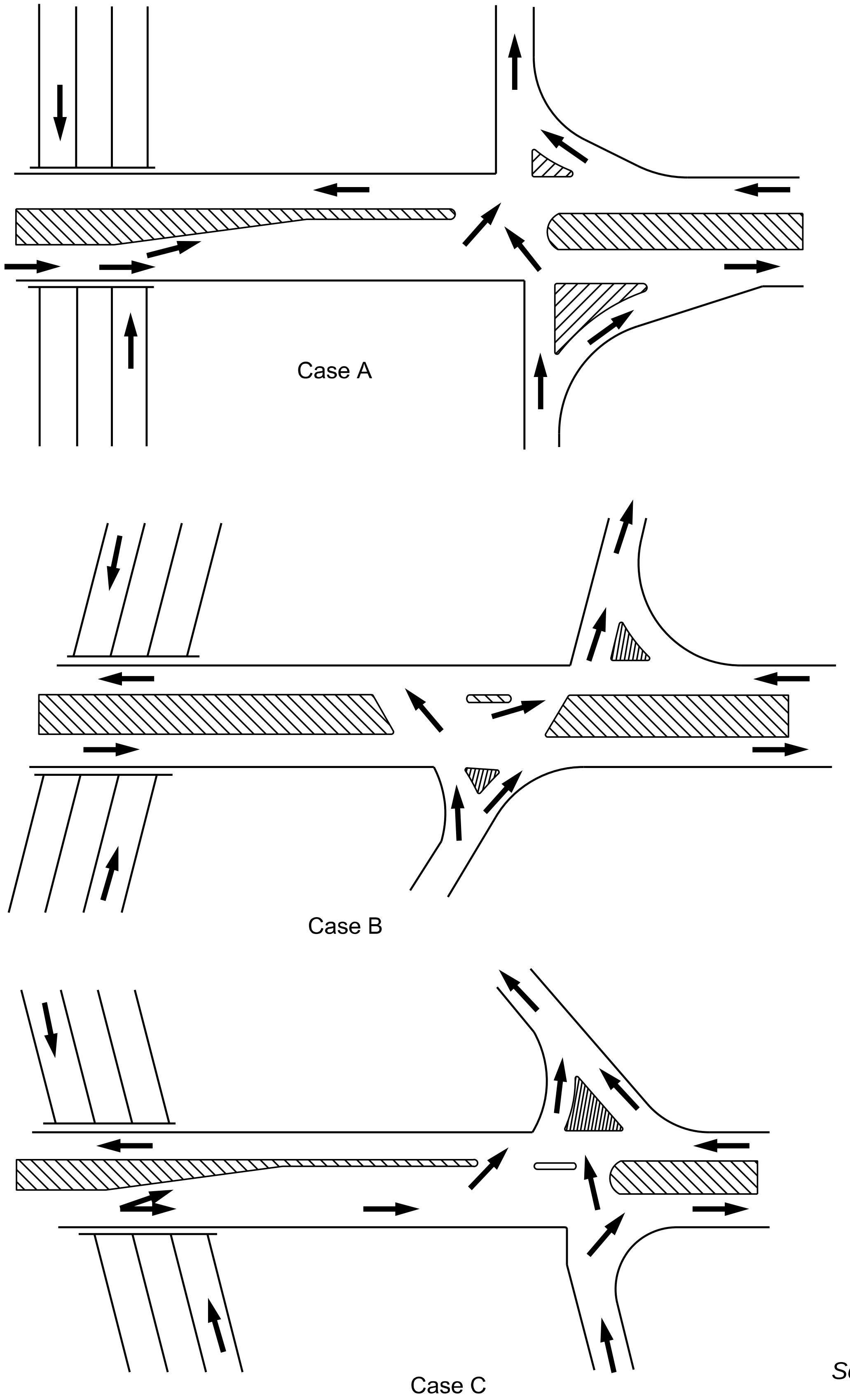
Providing access control along the crossroad of an interchange is an important design feature for both the safety and efficient operation of an interchange. The access control line is defined as a line that restricts direct access to property abutting the roadway. See Abu Dhabi *Access Management Policy and Procedures Manual* (21) for additional details on access control limits at interchanges.

Figure 12-60: Crossroad Designs to Discourage Wrong-Way Entry (Two-Lane)



Source: (1)

Figure 12-61: Crossroad Designs to Discourage Wrong-Way Entry (Divided)



Source: (1)

Table 12-16: Ramp Continuous Frontage Road Intersection

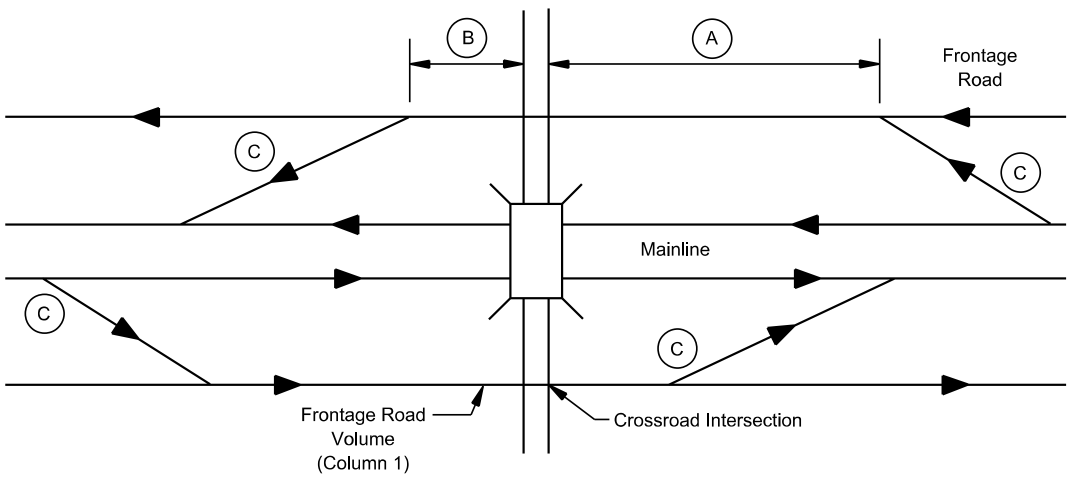
Frontage Road Volume at Intersection (vph)①	Exit Ramp Volume (vph)②	"A"		
		Typical Minimum	Typical Desirable	Special Considerations
200	140	115 m	150 m	80 m
400	280	140 m	170 m	110 m
600	420	150 m	190 m	120 m
800	560	165 m	210 m	130 m
1000	700	180 m	230 m	140 m
1200	840	195 m	265 m	145 m
1400	980	210 m	295 m	150 m
1600	1120	235 m	325 m	160 m
1800	1260	260 m	360 m	170 m
2000	1400	295 m	395 m	180 m

① *Assumes the total volume of traffic on the frontage road including the merged exit-ramp volume.*

② *Assumed to be 70% of total volume in first column.*

Notes:

1. *Table values are acceptable for planning purposes only. Final lengths will be based on a detailed operational analysis. This design only should be used in restricted urban areas.*
2. *Distance "B" is determined on a case-by-case basis.*
3. *"C" is a slip ramp.*



12.8 Ramp/Freeway Divergence and Convergence

12.8.1 Divergences

Where two freeways separate, provide a divergence as shown in Figure 12-62. The most important concept in the use of a major divergence is that if the route turns at an interchange, the physical divergence of the roadways should also occur in the same direction. To maintain lane balance, an additional interior lane will be required preceding the divergence. The widening of the interior lane from 3.65 m to 7.3 m should occur in a distance of 300 m to 540 m. This provides a driver in the centre lane the option of selecting either direction of travel without having to change lanes.

Add additional lanes to the side of the lesser-preferred route. Check for lane balance. Pavement joints should normally favour the freeway with the higher volume of traffic.

Where a divergence is required but the preferred design of an equal split of the roadways cannot be achieved due to the existing freeway alignment, a modified divergence design can be used as shown in Figure 12-62(d). In addition, where a divergence design is required and sufficient right-of-way is not available to build the one-sided divergence and where the diverging traffic volume is not significant, a two-lane exit terminal design may be considered.

12.8.2 Convergences

Where two freeways merge, provide a convergence design as illustrated in Figure 12-63. The number of lanes downstream from the convergence generally will be one less than the combined total of the two approaching roadways. In Abu Dhabi, the number of lanes departing the merge area be the same number as the two approaching roadways (i.e. do not merge the two inside lanes together).

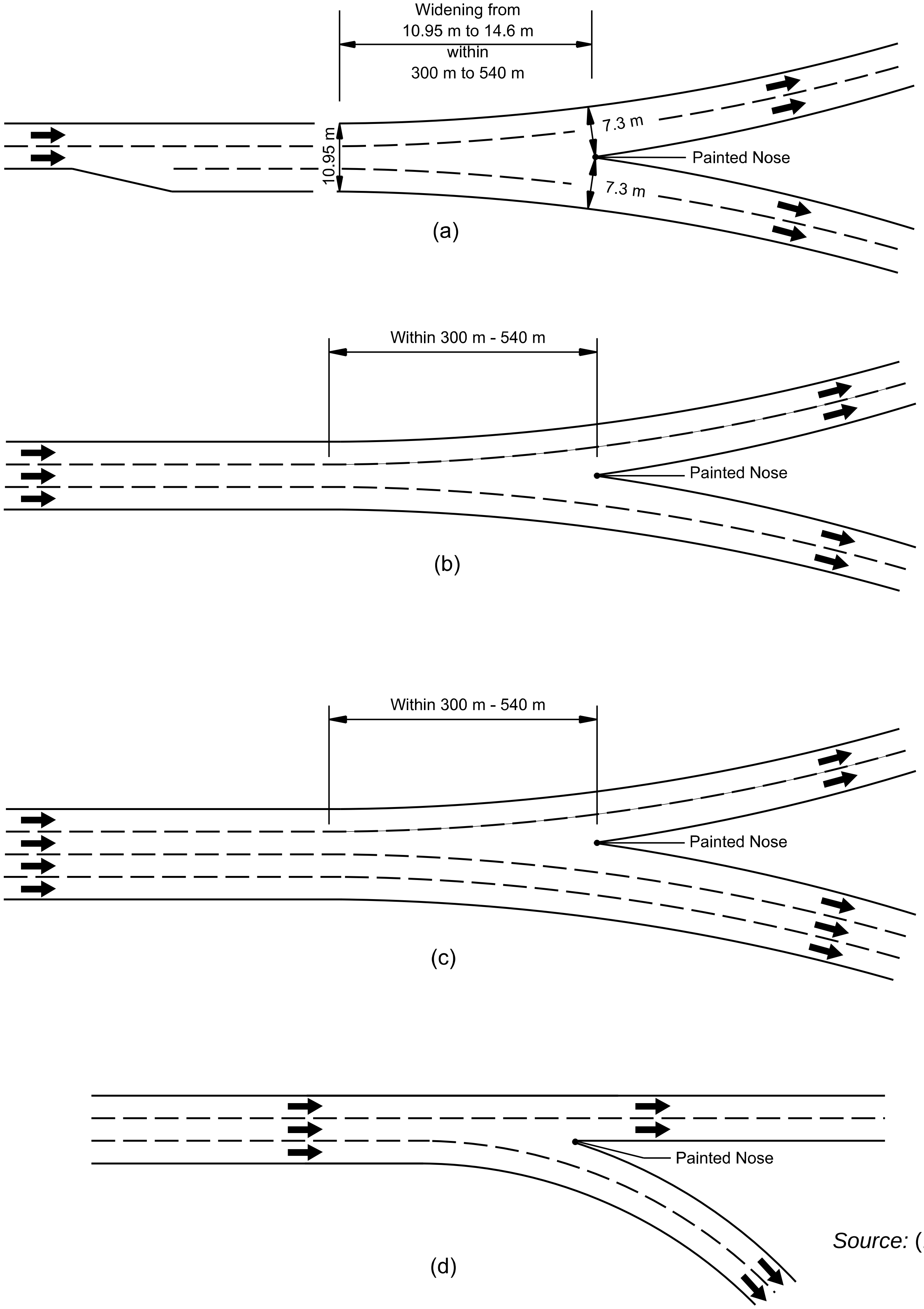
Typically, a lane drop will be required downstream from the convergence. Drop the right slow-speed lane versus the left high-speed lane.

12.8.3 Manoeuvre Areas

The diverging and merging manoeuvre area is a portion of the speed-change lane contiguous with the through travelled way. As shown in Figure 12-62 and Figure 12-63, the length of manoeuvre area is measured for tapered terminals from a half lane width to the diverging or merging tip (painted nose). For the parallel-lane terminals, it is considered to be from the beginning or ending point of the full auxiliary lane. The configuration of the ramp terminal thus has a dual function, to accommodate the necessary speed change and to allow ramp traffic to effectively separate from or merge with traffic on the freeway.

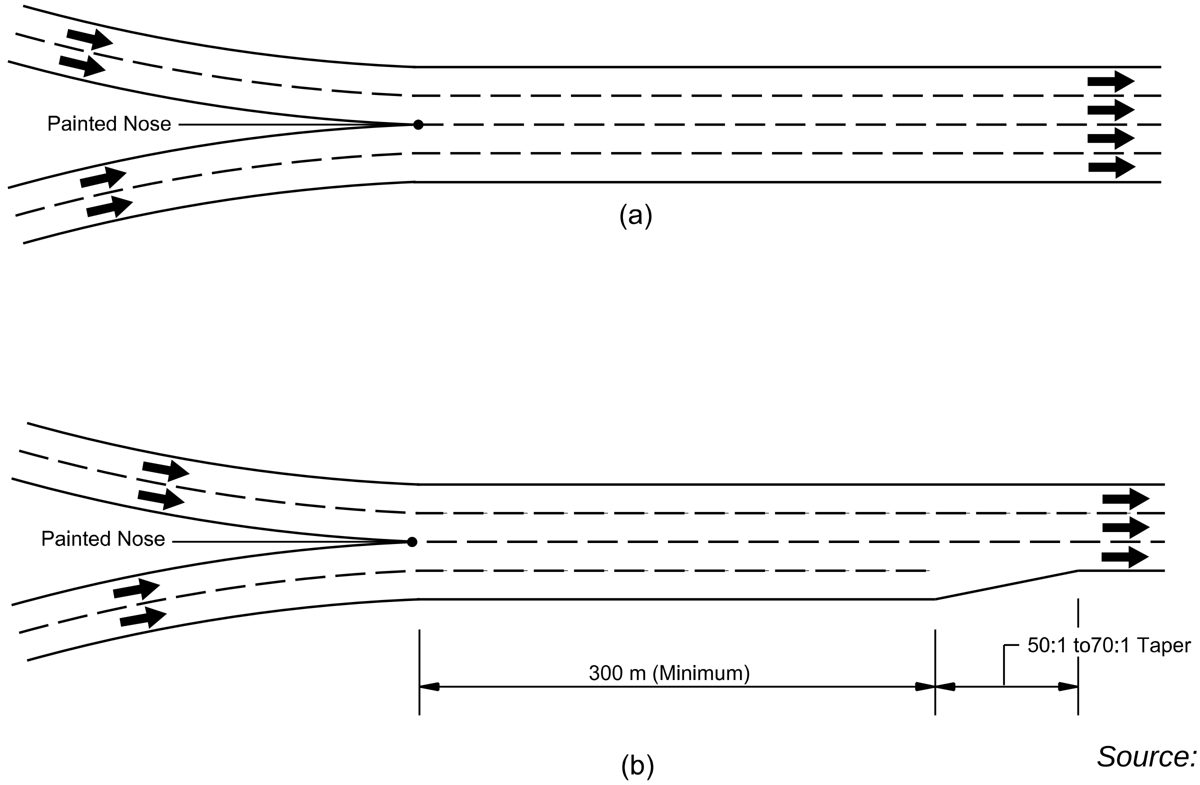
At entrance terminals, the merging manoeuvre area allows entering drivers to select and accept a gap in the adjacent traffic stream. Entering drivers must travel at near the speed of the adjacent flow and match an acceptable gap before reaching the end of the manoeuvre area. Some assistance comes from drivers in the right through lane, who anticipate a vehicle entering from the ramp and adjust their speed or shift to a lane on the left. However, as traffic builds up acceptable gaps become less frequent and longer manoeuvre areas are necessary.

Figure 12-62: Major Diverges



Source: (1)

Figure 12-63: Major Converges



Source: (1)

When exit terminals are appropriately designed for speed change, diverging traffic has little impact on through traffic during free-flow conditions (low-volume periods). Here exiting vehicles separate at the prevailing freeway speed with little effect on through traffic in the outer lane. However, during relatively high-volume and near-capacity conditions on the exit terminal, density builds sufficiently to impact the outer lane and possibly other lanes on the freeway. The provision of a diverging manoeuvre area tends to mitigate this problem. By lengthening the manoeuvre area, particularly as part of an auxiliary lane, a significant increase in efficiency can be achieved.

Lengths of manoeuvre areas for tapered terminals are in the range of 90 m to 120 m for diverging, and 150 m to 210 m for merging. For designs with a parallel lane, these dimensions are generally 30 m longer. The indicated lengths should be part of the ramp terminal design. See the ITE *Freeway and Interchange: Geometric Design Handbook* (36) for additional guidance.

12.9 HOV and Public Transport

High occupancy vehicle (HOV) facilities and public transportation have been implemented in rural and urban freeway corridors for several decades. These improvements are a means to increase the person carrying capacity of the freeway corridor.

Some of the design and operational treatments that are preferred today include barrier-separated one- and two-lane HOV facilities, managed lanes, and high occupancy toll (HOT) lanes. As part of these design concepts, separate interchanges can be used to provide access to and from these special facilities, park and ride lots for cars and vanpools, and freeway express buses.

As appropriate, transit may be incorporated into the freeway corridor. This can result in a significant increase in person travel. With most existing freeways, additional right-of-way would be required whether the transit line is located with the median or adjacent to the freeway facility. With transit, park-n-ride lots should be considered as well as strategically located stations that may also serve as mode transfer facilities. HOV and transit facilities are discussed in Section 7.7.4.

12.10 Toll Road Facilities

Toll plazas are most commonly found in various locations depending on where the toll authority elects to collect tolls. These toll collection locations are at toll plazas unless electronic toll collection (ETC) is the only method of payment and then a gantry with ETC equipment may be all that is present. The most common location for the placement of a toll plaza is at the crossing on one or both directions of travel on the mainline or ramp. Other locations of toll collection are on collector-distributor roads, toll bridges, approaches to toll ferries and in a similar sense at airports at the parking exit.

The primary elements of a toll plaza include:

- approach zone (diverge taper);
- departure zone (merge taper);
- queue area;
- toll island/barrier;
- recovery area;
- toll collection point; and
- protective canopy.

See the ITE *Freeway and Interchange Geometric Design Handbook* (36) for guidance on toll facilities.

13 PEDESTRIAN AND CYCLIST FACILITIES

13.1 Overview

When planning transportation improvements, the designer needs to consider the travel needs of all users of a transportation corridor, including cyclists and pedestrians. Cyclists and pedestrian travel demand near a project should be determined early in the project-planning phase. When sufficient demand is indicated, the project should include the appropriate accommodations.

The application of the guidelines presented in this chapter will result in consistent designs and suitable roadway design changes that will facilitate cyclists and pedestrian travel. Such changes will provide improved transportation opportunities for both cyclists and pedestrians.

13.2 Pedestrians

13.2.1 Public Realm

The public realm includes all exterior places, linkages and built form elements that are physically and/or visually accessible regardless of ownership. These elements can include, but are not limited to, streets, pedestrian ways, cycle paths, bridges, plazas, nodes, squares, transportation hubs, gateways, parks, waterfronts, natural features, view corridors, landmarks and building interfaces.

To simplify and plan for the Emirate, the public realm is organised into four categories: Parks, Streetscapes, Waterfronts, and Public Places. Definitions for these categories are as follows:

1. Parks. Public spaces within a community for recreational use. Parks may include natural areas (e.g. mountain ridges and wadi systems).

2. Streetscapes. The visual elements of a street including the road, sidewalk, street furniture, trees, and open spaces that combine to form the street's character.

3. Waterfronts. All land areas along the water's edge.

4. Public Places. All open areas within a community visible to the public or for public gathering or assembly.

Where used in this *Manual*, the term "pedestrian realm" refers to streetscapes. For guidance on other public realms, see the Urban Planning Council *Public Realm Design Manual* (13).

13.2.2 Pedestrian Realm Zones

Property planned pedestrian realms and pathways are essential in providing pedestrian mobility, safety, and accessibility, particularly for persons with disabilities, children, and older adults. They reduce the incidence of pedestrian collisions, injuries, and deaths in residential areas and along roadways. They separate pedestrians from traffic. The presence of sidewalks was cited in a study as the one physical factor in the roadway environment with the greatest effect on pedestrian safety in residential areas.

There are four primary zones in the pedestrian realm: Frontage, Through, Furnishings, and Edge. Because interaction occurs between these zones, development of a cohesive design for the pedestrian realm is important. Designer must consider the unique conditions associated with each zone as well as how the pedestrian realm interacts with other elements of the street. Maintaining clear sight lines between pedestrians, cyclists, and motorists in these areas of interaction is critical.

A typical pedestrian realm is provided in Figure 13-1. For guidance on designing these areas, see Chapter 9 “Urban Streets” and DMT *Public Realm Design Manual* (13). In some cases, a cycle track may be located within the pedestrian realm. See Section 13.2.4 for design of cycle tracks and other cycle facilities. Figure 13-2 shows desirable pedestrian realms.

13.2.2.1 Frontage Zone

The Frontage zone is adjacent to the building or property line. It is the space between the building façade, wall or fence and the Through zone.

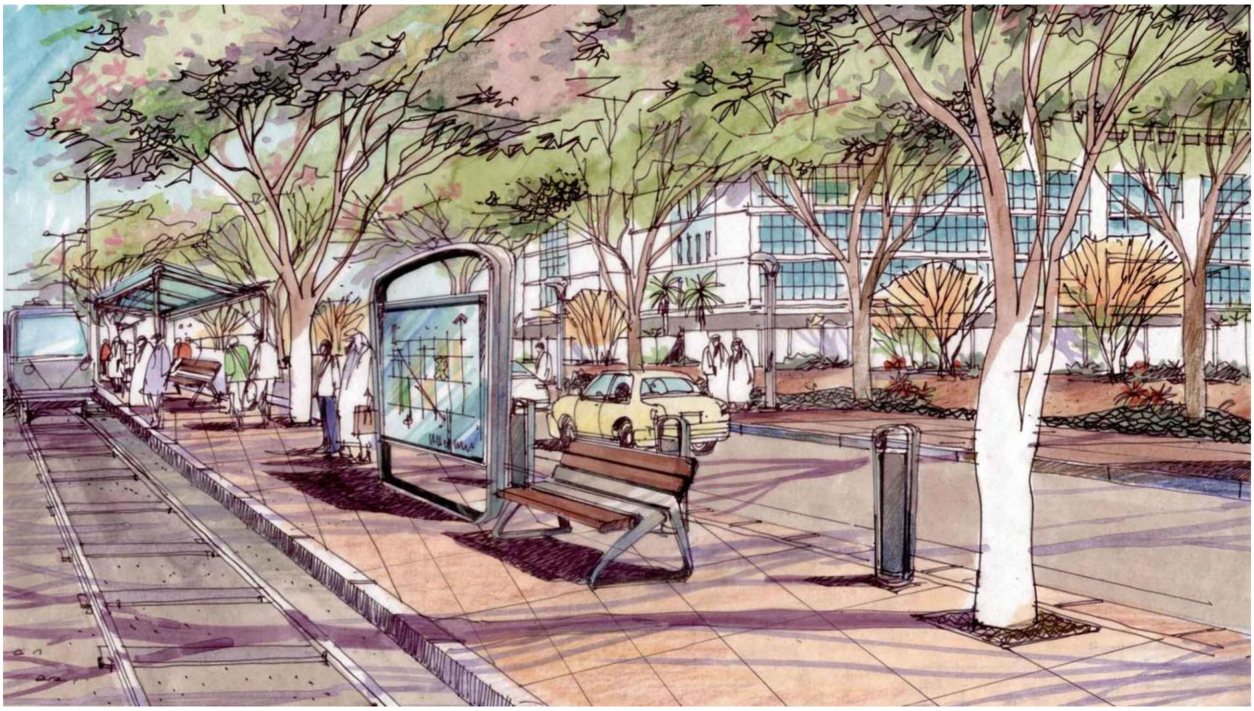
- Keep this space as clear as possible so that people may walk and stand in the shadow of buildings.
- Vertical changes between the pedestrian realm grade and ground floor levels should be addressed internally within buildings/plots.
- Construct the Frontage zone at the same grade and level as the Through zone.
- The surface material should be the same as the Through zone, but accent paving or colour may be used to delineate the Frontage zone.

Figure 13-1: Pedestrian Realm Elements

Frontage Through Furnishings Cycle Track Edge

Source: (4)

Figure 13-2: Pedestrian Realms



13.2.2.2 Through Zone

The Through zone is an obstacle-free space for pedestrian movement. This is the primary walking area of the pedestrian realm. It must remain both horizontally and vertically clear and provide a direct connection along pedestrian desire lines.

- Provide a firm, smooth, slip-resistant surface.

- Increase the width of the Through zone in places that will attract high volumes of pedestrians, such as near public transit facilities, malls, and other major destinations to accommodate pedestrian movement and volumes.

13.2.2.3 Furnishing Zone

The Furnishings zone is where street furniture, necessary utility equipment, trees, landscaping, transit stops, and other features (e.g. kiosks, sidewalk cafés, vendors) may be located. This is primary the buffer space between the active pedestrian walking area of the Through zone and adjacent thoroughfares.

- Consolidate and organise furnishings to maximise public use and benefit.
- Break up the Furnishings zone to provide pedestrians access to crossings, taxi lay-bys, bus stops, and other facilities.
- Provide screening and buffering of utility fixtures in this zone while maintaining clear access for utility providers for maintenance.
- See the DMT *Public Realm Design Manual* (13) for additional guidance related to streetscape furnishings.

13.2.2.4 Edge Zone

The Edge zone is adjacent to on-street parking or travel lanes. The Edge zone provides space to open a car door. It is where pedestrians wait for taxis or buses. This zone is often where street lights, signals, traffic signs, parking metres, and other street-related infrastructure are placed. These elements may also be placed in the Furnishings zone, particularly on narrower streets, or on the side median where there is a frontage lane.

- Provide 4.5 m vertical clearance in travel lanes adjacent to the Edge zone for tall vehicles.
- Combine Furnishings zone and Edge zone where necessary for transit stops and taxi lay-bys. If this is not possible, provide a minimum of 1.5 m horizontal clearance where pedestrians are likely to wait for taxis or buses.

13.2.2.5 Cycle Track

Designated track for cyclists; may not be required on some streets. A cycle track may be included in the pedestrian realm, between the Furnishings and Edge zones, to provide cyclists with a dedicated right-of-way separate from the travelled way. Coordination is required with the client to determine the specific facilities required.

13.2.2.6 Guidelines for Sidewalk Installation

All urban roadways should have some type of walking facility out of the vehicular travelled way included in the initial construction. A separate walkway is much preferred. In rural areas, the walkway should be located near the right-of-way line and beyond the swale. In extreme cases, a rural roadway shoulder can also provide a safer pedestrian accommodation than walking on the travel lanes themselves.

The purpose of pedestrian realms is to provide direct connections between destinations to form an unbroken coordinated pedestrian network. It is usually not difficult to ascertain where connections will be required in the early stages of development. This will prevent the later construction of circuitous routes. Many routes in urban subdivisions require pedestrians to walk out of the subdivision onto a main road and then to travel parallel with the major road network to arrive at an activity centre. This can result in a walking distance five times that of a direct route.

Easements permitting pedestrian access through the middle of residential blocks can provide a direct connection for pedestrians with school and commercial needs. High volume streets near schools should be provided with sidewalks separated from the roadways themselves to help to improve the safety of walking children. Parking should be closely regulated and designed to maximise sight distance for children and to eliminate hazards. Placement of schools within neighbourhoods, as opposed to placing them on the Boulevards and Avenues can eliminate walking trips along and across these facilities.

The actual construction of sidewalks, however, can be provided for when development progresses enough to generate pedestrian demand. Well-designed sidewalk routing is best accomplished by concurrent planning of commercial and residential development within an area. For additional guidance, see the DMT Public Realm Design Manual (13) and the Abu Dhabi “Urban Street Design Manual” (USDM).

Sidewalk pavers should be made of high quality material with low water absorption and decorative stone chips, rather than relying solely on pigmentation. Obtain approval from DMT for their tiling strategy for selection of sidewalk pavers. Slip resistant finishes are expected on sloped surfaces (6% or greater). Longitudinal ramps (pedestrian ramps) should not exceed a maximum of 8.3% (1 in 12) gradient. Pedestrian crosswalk ramps shall be located at all intersections and all other locations where main pedestrian traffic crosses kerblines. The minimum sidewalk cross slope should be 1.5% towards the roadway.

Sidewalk Widths - The guidelines in Table 13-1 should be used to determine sidewalk width. Also refer to Figure 13-1: Pedestrian Realm Elements for guidance.

Table 13-1: Sidewalk Width Guidelines

Area	Vicinity	Width Range (m)
City Context	Boulevard	2.8 - 4.0
	Avenue	2.4 – 4.0
	Street	2.4 – 3.0
	Access Lane	2.0 – 2.5
Town Context	Boulevard	2.4 – 3.5
	Avenue	2.0 – 3.0
	Street	2.0 – 2.4
	Access Lane	2.0 – 2.5
Commercial Context	Boulevard	2.4 – 3.5
	Avenue	2.0 – 3.0
	Street	2.0 – 2.4

	Access Lane	2.0 – 2.5
Residential Context	Boulevard	2.0 – 3.5
	Avenue	2.0 – 3.0
	Street	2.0 – 3.4
	Access Lane	2.0 – 3.4
Industrial Context	Boulevard	2.0 – 3.6
	Avenue	2.0 – 3.4
	Street	2.0 – 3.0
	Access Lane	2.0 – 2.5
Notes: • Use 3.5m if buses use kerb lane as part of a regular transit route. • Dimensions provided are assuming parallel parking.		

Further technical guidelines concerning the design of pedestrian facilities is given below:

- Ensure sidewalks are a minimum 2m wide (obstacle free), desirable cross slope to be 1.5% and maximum cross slope to be 3%.
- Paving should be designed, installed and maintained to be smooth and level. Surfaces should not be interrupted by steps or abrupt changes in level of more than 6mm. Changes in sidewalk level due to various building entrance levels (for the existing buildings) should be fixed or mitigated by engineering solutions.
- Longitudinal ramps (pedestrian ramps) should not exceed a maximum of 8.3% gradient.
- Provide pedestrian ramps at access locations to underground parking facilities (if not provided by the building owner) in order to avoid steps for pedestrians.
- Vertical separation of pedestrian paths from vehicle travel ways should be maintained. Typical kerb height for Boulevards, Avenues, Streets, Access Lanes and Frontage Lanes to be 10cm.
- Sidewalks should be separated from travelled ways by a minimum of 0.5m paved edge zone and 1.5m landscape zone. Where possible sidewalks along the travelled way edge should be eliminated and shifted to the building frontage while retrofitting streets as per the USDM.

13.2.2.7 Sidewalk Maintenance

Local agencies should have guidelines in place for the maintenance of pedestrian facilities. These general maintenance requirements should include a regular program of inspection.

13.2.3 Design Guidance

Pedestrians require movement not only along the street, but also at interaction areas where multi-mode user movements cross. As pedestrians are the most vulnerable of all street users, special care and consideration is needed to identify potential issues and to design facilities accordingly.

13.2.3.1 Pedestrian Level of Service

True pedestrian level of service is subject to a long list of variables, each of which has complex interactions with the others. One measure included in the *Highway Capacity Manual 2010* (17) is pedestrian crowding, which is useful at specific locations like rail terminals and stadiums, but is not generally used in the Emirate, because a modest amount of pedestrian crowding is helpful for personal security and urban vitality. The guidance for determining pedestrian walking and waiting level of service is provided in Section 9.3.2.2.

13.2.3.2 Pedestrian Realm Widths

Table 13-2 provides widths for various zones within the pedestrian realm.

Table 13-2: Width for Pedestrian Realm Zones

Street Family	Frontage		Through		Furnishing		Cycle track(2)		Edge(1)	
	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max
City Context										
Boulevard	0.5	1.5	2.8	4.0	1.2	3.5	1.5	3.0	0.2	2.0
Avenue	0.5	1.5	2.4	4.0	1.0	3.0	1.5	3.0	0.2	2.0
Street	0.5	1.5	2.4	3.0	1.0	2.4	1.5	3.0	0.2	2.0
Access lane	n/a	n/a	1.8	2.5	n/a	n/a	n/a	n/a	0.2	1.5
Town Context										
Boulevard	0.5	1.5	2.4	3.5	1.2	3.0	1.5	3.0	0.2	2.0
Avenue	0.5	1.5	2.0	3.0	1.0	2.4	1.5	3.0	0.2	2.0
Street	0.5	1.5	2.0	2.4	1.0	2.0	1.5	3.0	0.2	2.0
Access lane	n/a	n/a	1.8	2.5	n/a	n/a	n/a	n/a	0.2	1.5
Commercial Context										
Boulevard	0.5	1.5	2.4	3.5	1.2	3.0	1.5	3.0	0.2	2.0
Avenue	0.5	1.5	2.0	3.0	1.0	2.4	1.5	3.0	0.2	2.0
Street	0.5	1.5	2.0	2.4	1.0	2.0	1.5	3.0	0.2	2.0
Access lane	n/a	n/a	1.8	2.5	n/a	n/a	n/a	n/a	0.2	1.5
Residential Context										
Boulevard	0.5	1.0	1.8	3.5	1.2	2.0	1.5	3.0	0.2	2.0
Avenue	0.5	1.0	1.8	3.0	1.0	2.0	1.5	3.0	0.2	2.0
Street	n/a	n/a	1.8	3.4	n/a	n/a	1.5	23.0	0.2	2.0
Access lane	n/a	n/a	1.8	3.4	n/a	n/a	n/a	n/a	0.2	1.5
Industrial Context										
Boulevard	0.3	0.5	2.0	3.6	1.2	2.4	1.5	3.0	0.2	2.0
Avenue	0.3	0.5	2.0	3.4	1.0	2.4	1.5	3.0	0.2	2.0
Street	0.3	0.5	2.0	3.0	1.0	1.5	1.5	3.0	0.2	2.0
Access lane	n/a	n/a	1.8	2.5	n/a	n/a	n/a	n/a	0.2	1.5

Optional

(1) Edge zone must be a minimum of 1.5 m where there is on-street parking or a cycle track. It may only go down to 0.2 m when sufficient room is available for signing, lighting, and utilities within an adjacent Furnishings zone.

(2) Desirable width of 1-way cycle track is 1.5m and the absolute minimum width of 1-way cycle track is 1.2m (for retrofitting projects only). desirable width of 2-way cycle track is 3.0m and the absolute minimum width of 2-way cycle track is 2m (for retrofitting projects only)"

Note: The dimensions in this table are for the design of atypical streets and are only to be used for specific conditions. All new streets should be designed using typical cross section dimensions in Section 9.5.

Total right-of-way using this Table shall not be less than the DMT Utility Corridors Design Manual (32) *absolute minimum.*

Source: (13)

13.2.3.3 Pedestrian Crossing Locations

Pedestrian crossings should be located at intersections and some mid-block locations where significant pedestrian movement is anticipated. To provide a high-quality pedestrian environment and to ensure pedestrian safety, the designer should provide pedestrian crossings on all urban streets to accommodate primary pedestrian desired lines. Desire lines are the most direct route to a destination that a pedestrian would like to take. The pedestrian crossing should meet the following:

1. Intersections. Provide crossings at all traffic-controlled intersection legs.
2. Desired Lines. Locate along desire lines to:
   - align with entrances to buildings, parks, walkways, etc.;
   - delineate preferred pedestrian routes; and
   - allow easily access transit stops.
3. Environmental Conditions. Crosswalk locations should depend on land use, pedestrian activity, traffic conditions, and urban form.
4. Walking Distances. Consider comfortable walking distances, especially during the harsh summer months, while determining the spacing between pedestrian crossings.
5. U-Turns. If U-turns are included, consider locating pedestrian crossings away from the U-turn movement to minimise conflicts with turning traffic.
6. Mid-Block Crossings. Figure 13-3 illustrate mid-block crossings. Table 13.3 provides the guidelines for installing pedestrian mid-block locations. Also note the following for mid-block crossings:
   - Locate mid-block crossings based on pedestrian movement, building entrances, attractions, traffic conditions, location of transit stops, urban form, etc.
   - Include overhead signage and lights to allow the driver to identify the crossing.
   - Provide curb extensions where there is on street parking to maintain pedestrian visibility.
   - Provide raised crossings where traffic calming is necessary.
7. Other Guidance. See the Abu Dhabi *Walking and Cycling Master Plan* (23) for additional guidance on pedestrian crossings.

13.2.3.4 Crossing Design

Pedestrian crossings can range from raised speed table style crosswalks to unmarked crossings.

Table 13-3 provides crossing criteria for various facility types, based on street family. Crossing and waiting areas should be free of any obstacles (e.g. street furniture and signs). Locate traffic signals and sign poles that are required for the crossing so as not to obstruct visibility for both pedestrians and drivers. Crossings can be raised or not, depending on specific conditions. Refer to the *Abu Dhabi Urban Street Design Manual* (4) for raised crosswalk design. Consider the additional following guidance on crossing designs:

1. Crossing Distance. The maximum uninterrupted crossing distance without a median refuge shall not exceed four travel lanes. To reduce crossing distances on existing streets, the designer should consider:

Figure 13-3: Mid-Block Crossings

Source: (4)

Table 13-3: Guidelines for Mid-Block Locations

Street Family	Vehicle ADT < 9,000			Vehicle ADT > 9,000 to 12,000			Vehicle ADT > 12,000 to 15,000			Vehicle ADT > 15,000		
	Speed Limit											
	≤30 km/h	40 km/h	60 km/h	≤30 km/h	40 km/h	60 km/h	≤30 km/h	40 km/h	60 km/h	≤30 km/h	40 km/h	60 km/h
City Context												
Street	●	●	N/A	●	●	N/A	●	●	N/A	●	○	N/A
Avenue	●	●	○	●	○		○	○				
Boulevard												

● *Acceptable to use marked uncontrolled crossing*
○ *Consider traffic calming, signal or other*
Do not use marked crosswalk without a signal and consider additional traffic calming measure.

Source: (4)

- providing curb extensions,
- narrow the width of travel lanes to the widths provided in Chapter 9 “Urban Streets,”
- reducing the number of travel lanes, and/or
- installing refuge islands located within clearly visible areas.

2. Curb Ramps. Provide curb ramps at each end to accommodate the change in grade at the ends of crossings. Place the curb ramp in line with the adjoining sidewalk. The pedestrian route should not diverge at a road of more than 1:5. See Section 13.2.3.9.

3. Grates and Manhole Covers. Ensure manhole covers, storm grates, and other obstacles that limit free movement.

4. Alignment. Mid-block crossings in medians on Boulevards and Avenues should be staggered so that pedestrians will be oriented towards oncoming traffic.

5. Width. The typical crosswalk width should be 3 m to 5 m on Boulevards with anticipated high pedestrian volumes or where shared with cyclists at major crossings.

6. Stop Bar. Place the vehicle stop bar 3 m ahead of the crosswalk.

7. Signalised Crossings. At signalised crossings, consider the following:

   - Signal type (fixed time, push button, etc.) and signal timing should be appropriate for all user requirements.

   - Provide enough green time to allow pedestrians to safely cross the street or intersection.

   - Where practical, use fixed-time traffic signals that allow pedestrians to cross with vehicles. This eliminates the need for pedestrian push buttons. Exceptions may be made at mid-block crossings.

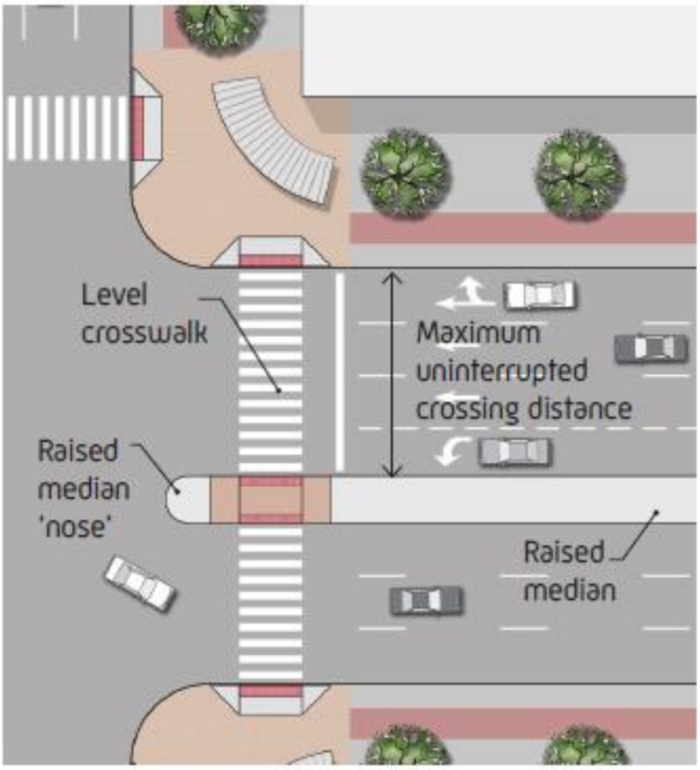
   - Provide clearly identifiable push buttons (or other type of activation device) adjacent to the crosswalk at all pedestrian activated crossovers.

   - Where pedestrians are a major factor, consider providing an exclusive pedestrian phase. Pedestrian crossing on Boulevards should be signalised.

- Provide dynamic timing (countdown) pedestrian signals.
- Provide audible pedestrian signals.
- Signalised pedestrian crossings should not have raised crosswalks, particularly on Boulevards. Signalised raised crosswalks can be considered for traffic calming and this should to be evaluated on a case-by-case basis.

8. Pedestrian Refuges. For wide crossings, it is desirable to provide a refuge area in the median to allow pedestrians to cross the road in two movements; see Figure 13-4. Consider the following:

Figure 13-4: Typical Crosswalk



Source: (4)

- The median width needs to be 2 m or wider and wide enough to store anticipated volumes of pedestrians, strollers, and cyclists. Typically, the refuge area should be 10 m^2 and on Boulevards 12 m^2.
- Drop curbs should be provided for the full width of the crossing.
- If the existing median width is 2 m or less, it is preferable to provide cut through refuge area versus installing two curb ramps.
- Crossing may be staggered or angled to orient pedestrians to oncoming traffic.
- Consider using fences/walls/screens to increase safety by managing pedestrian circulation and preventing vehicle conflicts.

- Consider using shade structures where trees cannot be used to shade crossings.

9. Raised Crosswalks. Raised crosswalks are speed tables marked for pedestrian crossing, built to a curb height of 100 mm. Along existing streets with curb heights of 150 mm or more, curbs should be dropped to the crosswalk height. Crosswalks should be at least as wide as the pedestrian crossing (3 m – 5 m), and preferably 4 m wide. Appropriate vehicle ramp grades desirably 6.67% (maximum 10% slope) should be considered for specific street types. shows a typical raised crosswalk. See Figure 13-7 & Figure 13-8 for typical details of raised crosswalks.

Figure 13-5: Raised Crosswalk

Source: (4)

10. Pedestrian Grade Separations : Pedestrian grade separations are not normally provided on roadways. However, if pedestrian use is extensive, an overpass or underpass may be considered. Justification for pedestrian grade separation structures derives from a detailed study of present and future community needs. Each situation should be studied separately and the study should include pedestrian generating sources, travel patterns, crossing volumes, roadway classification, location/circuitry of adjacent crossings, land uses, sociological and cultural factors, and the predominant type and age of users.

    Established pedestrian patterns should be maintained across primary routes. Separate pedestrian structures should be provided if vehicular crossings are inadequate for pedestrians. If a circuitous route is involved, a pedestrian separation may be justified. Special consideration should be given to school crossings.

    The choice between an overpass or underpass should be based on relative costs, groundwater influence, drainage, existing utilities, current and future land use, visibility, topography and the surrounding architecture.

11. Pedestrian Underpasses: Underpasses require special consideration due to visibility issues and the potential for criminal incidents and vandalism. If an underpass is used, unobstructed visibility shall be provided through the structure and approaches. The desired vertical

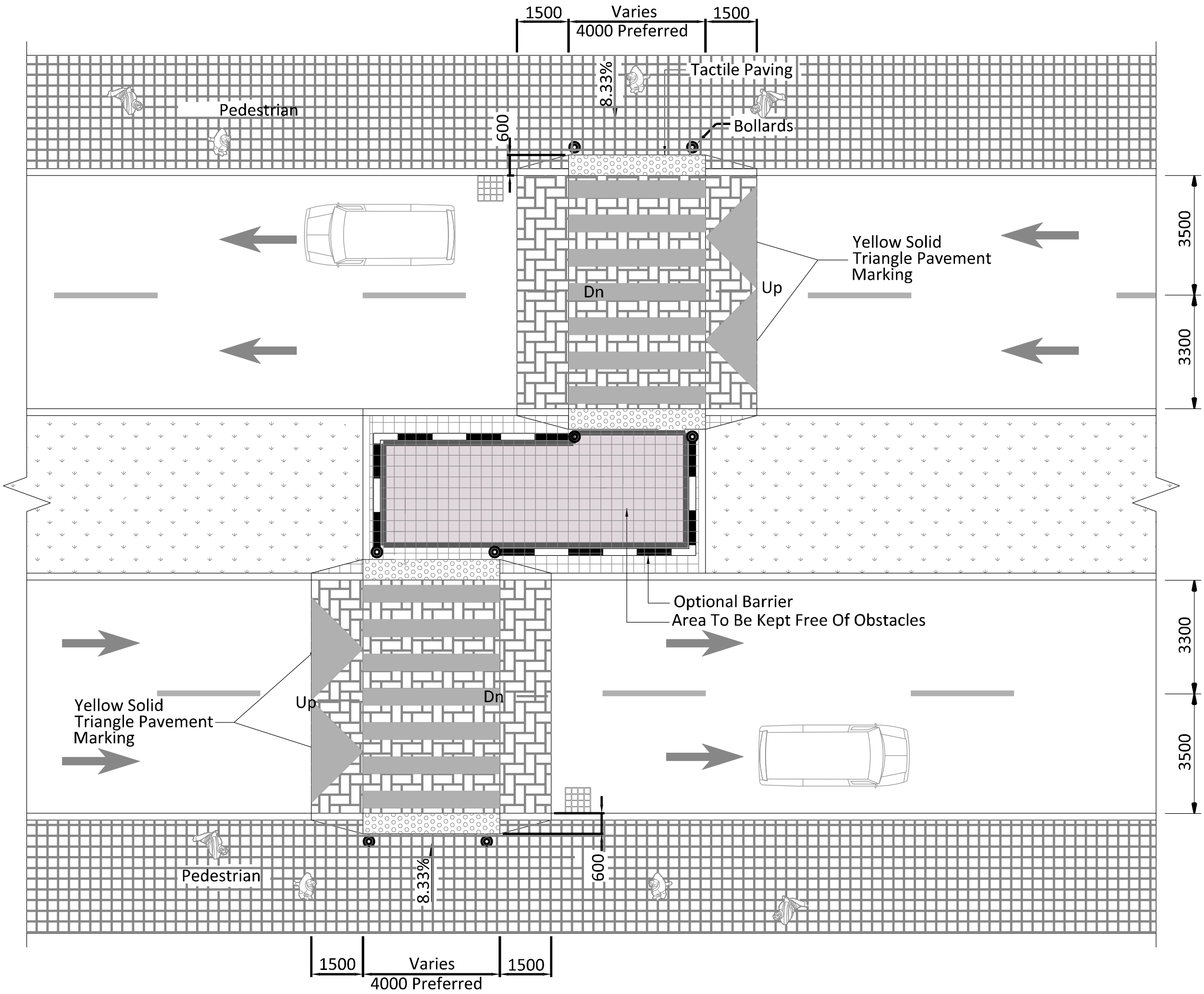
clearance is 3.0m, but in no case shall the clearance be less than 2.0m. The minimum width shall be 2.5m.

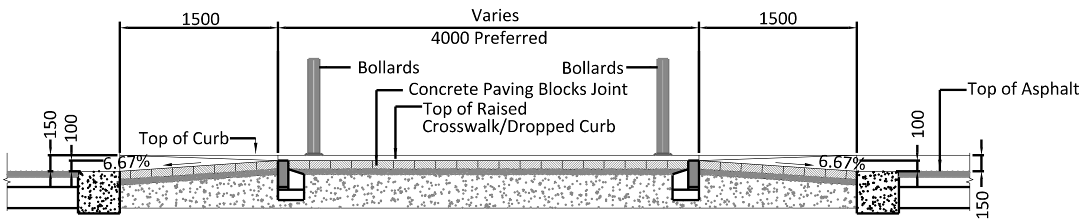
13.2.3.5 Driveway and Off-Street Loading Design

Design driveways and off-street loading access to ensure pedestrians have the right-of-way over motor vehicles. Design the driveway so vehicles are required to change grade, not pedestrians. Refer to the client regulations and guidance for off-street loading requirements. Other guidance for driveway designs includes:

- Orient driveways at 90 degrees (right angles) to roadway. Bend the drive lane if necessary.
- Design driveways as ramps, not as minor intersections.
- Ramp driveways up to pedestrian realm level. Continue pedestrian realm treatment across driveway.
- The maximum entry speed for turning vehicles should be 15 km/h.
- The maximum driveway width should be 7 m. This may be increased for industrial driveways where trucks are generally the turning vehicle.
- Control vehicles with stop and yield signs or with traffic signals.
- Minimise the number of driveways on Boulevards and Avenues; see the Abu Dhabi *Access Management Policy and Procedures Manual* (21).

Figure 13-6: Typical Raised Crosswalk (Avenue)



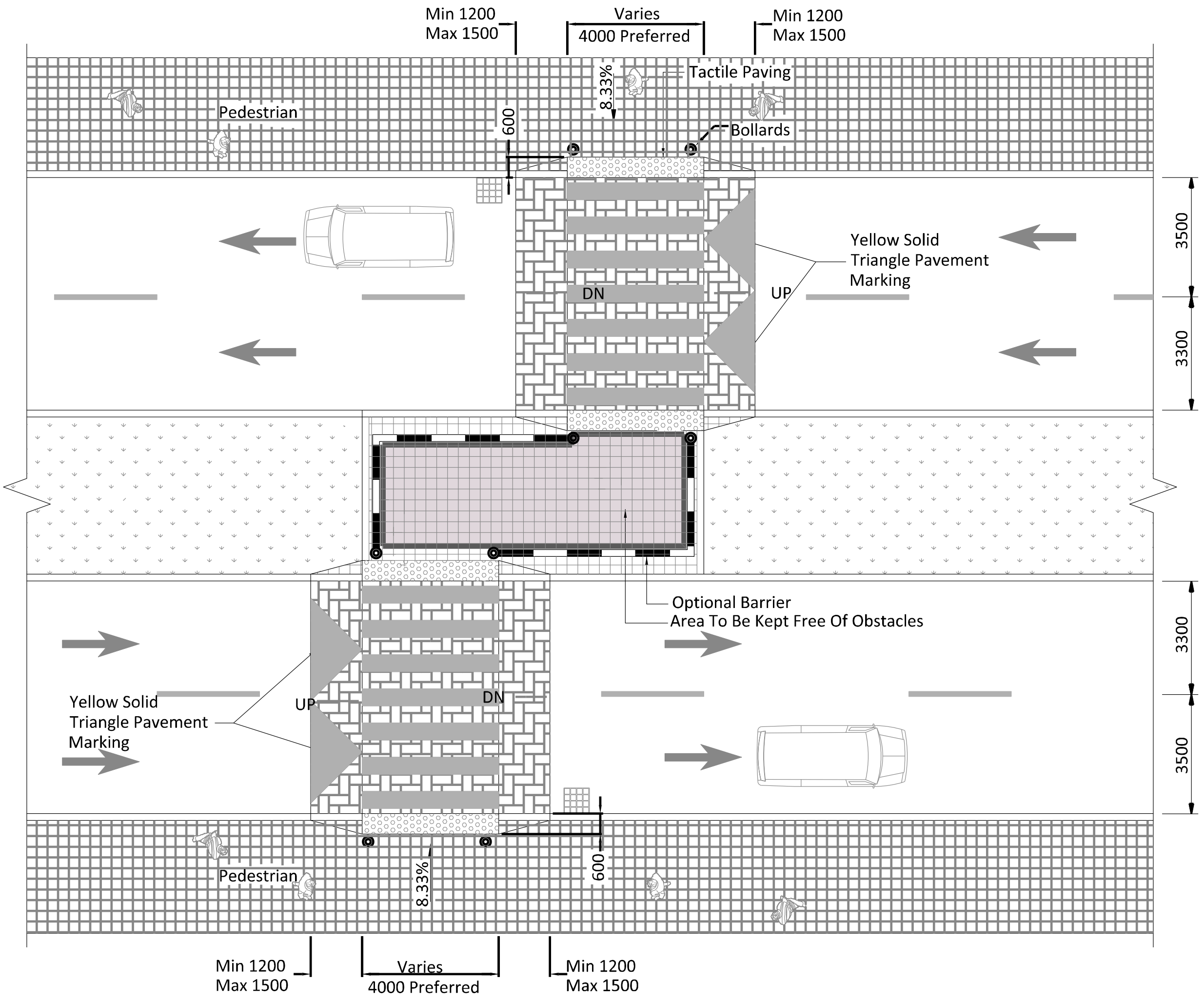


Cross Section

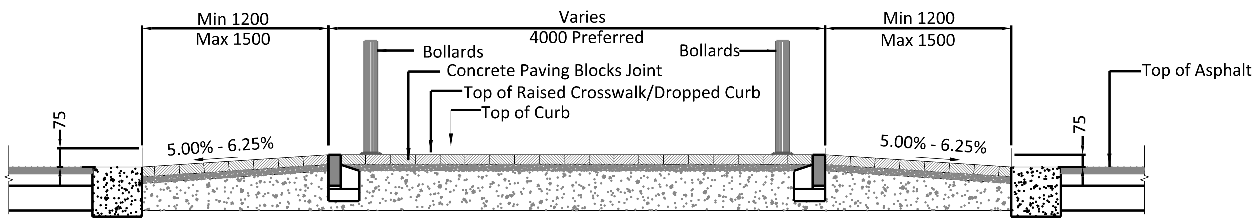
It is to be noted that the current practice in Abu Dhabi is to use a single bollard in the middle instead of two bollards on either side as shown above.

Source: (4)

Figure 13-7: Typical Raised Crosswalk on Bus Route (Avenue)



Plan



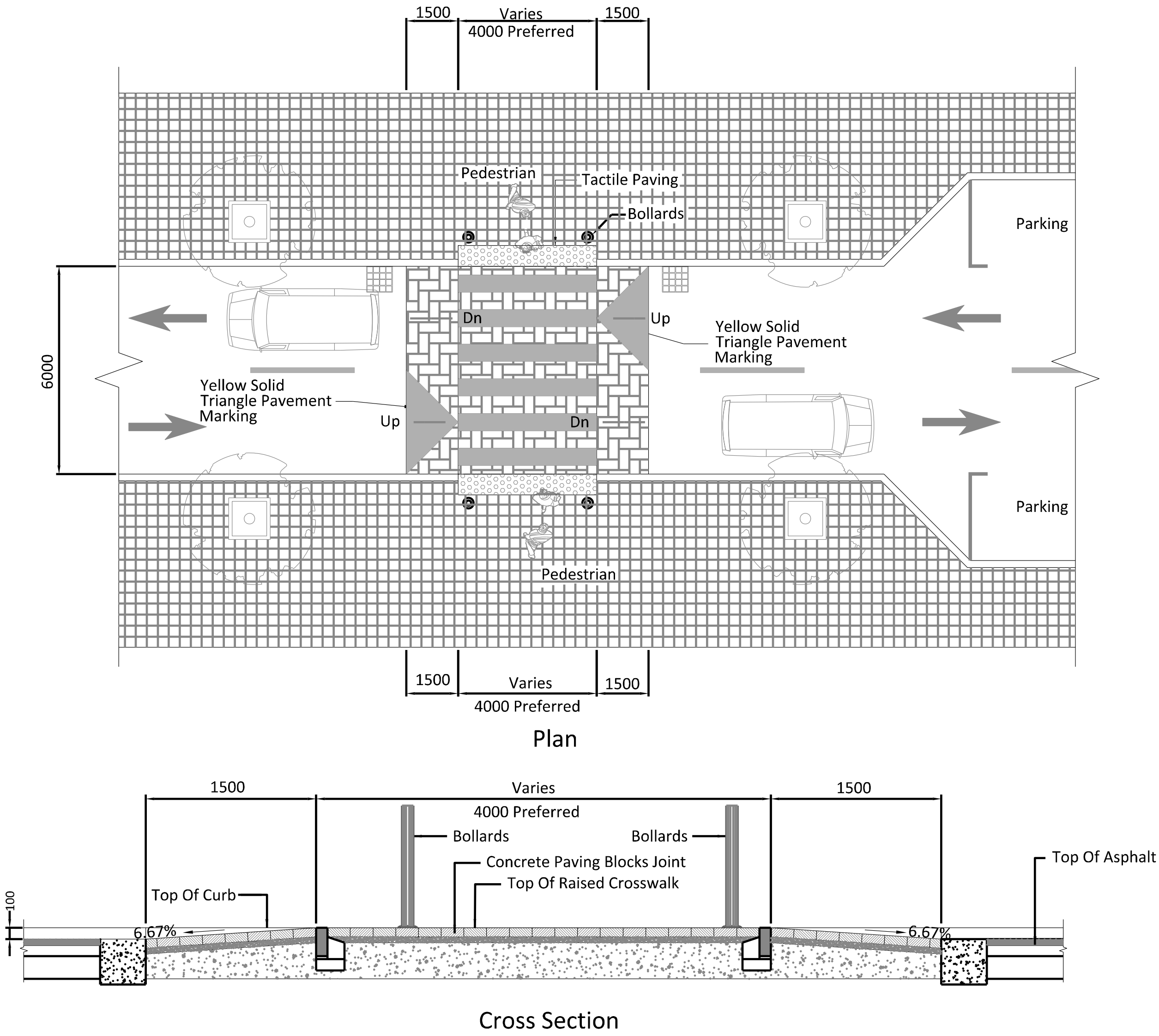
Cross Section

Note: Due to local conditions, height may be from 7.5 cm to 15 cm based on Authority's discretion and approval.

It is to be noted that the current practice in Abu Dhabi is to use a single bollard in the middle instead of two bollards on either side as shown above.

Source: (4)

Figure 13-8: Typical Raised Crosswalk (Street)



It is to be noted that the current practice in Abu Dhabi is to use a single bollard in the middle instead of two bollards on either side as shown above.

Source: (4)

13.2.3.6 Universal Access Guidelines

The mobility needs of persons with disabilities must be considered in the decision to locate and install sidewalks. The following identifies several design requirements for implementing universal access:

- Shall conform to international best practice in universal access.
- Shall locate lighting, signposts, refuse/recycling containers, trees, bollards, benches/ furniture/fixtures, etc., at or beyond the boundaries of pedestrian routes.

- Shall create a clear distinction between pedestrian routes and adjoining surfaces using visual indicators and tactile paving.
- Shall use a maximum gradient of 1:20 on all pedestrian routes; gradients above 1:20 shall use steps with integrated ramping and be clearly identifiable and contrast visually with their surroundings.
- Shall place gratings so that the long dimension is perpendicular to the dormant direction of travel.
- Shall provide curb cuts at right angles to path of travel with flared non-slip sides.
- Shall use curb cuts that are of a clearly different and detectable texture.
- Shall construct traffic islands with materials and finishes that are easily distinguishable from the surrounding paving.
- Shall use traffic islands that are a minimum width of 2.0 m to provide persons using mobility aids and seniors with safe resting zones.
- Shall use audible signals that are a minimum 15 dB louder than ambient noise.
- Shall provide two different audible signals identifying when it is safe to cross the street.
- Shall provide a minimum of 4% (Refer to Table 16-1) reserved parking facilities for disabled access with minimum dimensions of (2.5m + 1.5m) x 5.5m.
- Shall provide the international symbol of accessibility on disabled access reserved parking.
- Shall incorporate Braille in all signage elements in all public places.
- Shall provide an accessible route from designated disabled access parking stalls to all accessible entrances
- Should locate disabled access parking near the primary circulation route'
- Should design seating arrangements to allow mobility restricted users to sit alongside friends and family or in groups.
- Should use well-defined edge treatments such as plant materials, change in texture or curbs to indicate extent or change in route.

13.2.3.7 Curb Height

Curbs shall be designed to discourage motor vehicles from encroaching onto the pedestrian realm while still making it easy for pedestrians to step up and down from the pedestrian realm to the travelled way. Consider the following:

- Typical preferred curb height is 100 mm for Boulevards, Avenues, Access Lanes and Frontage Lanes.

- Use of rolled curbs are appropriate to minimise sand accumulation, but should only be used in areas with low traffic volumes.

- Higher curbs may be installed at bus stops.

- Where parking on curbs in the pedestrian realm is an issue, employ pedestrian protective techniques (e.g. bollards and planters) instead of higher curbs.

- Provide for positive drainage via swales, cross slopes, longitudinal grades, and other grading techniques in place of higher curbs.

13.2.3.8 Slopes and Grade

Provide a desirable cross slope of 1.5% to 2% on all paved surfaces in the pedestrian realm and street crossings (including sidewalks and ramps) with a maximum up to 3%. Maintaining this cross slope will facilitate travel by wheelchair users, minimise tripping hazards for pedestrians, and provide positive drainage for hard surfaces. The designer should note the following:

- Longitudinal grades in the pedestrian realm shall not exceed a maximum of 1:20. If the grade is larger anywhere in the pedestrian realm (e.g. along a building frontage), provide a longitudinal ramp.
- Longitudinal ramps should not exceed a maximum ramp grade of 1:12.

- Provide edge protection for ramps steeper than 1:20 or landings more than 1.3 m above the adjacent grade.

13.2.3.9 Curb Ramps

Curb ramps provide accessibility at intersections, mid-block crossings, and other areas where elevated walkways are edged with curbing. Curb ramps facilitate crossing for wheelchair users, people pushing strollers, cyclists, and others. They help sight-impaired pedestrians identify the street crossing location. The designer should note the following:

- Two curb ramps per corner should be provided at all intersections, one in the direction of each crosswalk.

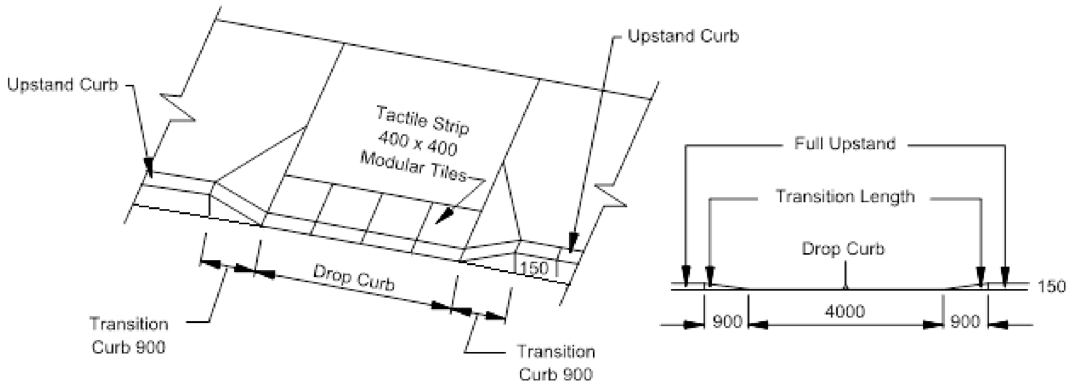
- Locate curb ramps in the centre of the crosswalk and construct them with the ramp the full width of the crosswalk.

Design the low end of the curb ramp so it meets the street grade with a smooth transition and no lip. See Figure 13-9 for design details on curb ramps.

- Curb ramps should not be used at channelization islands or median refuge islands that are 2 m or less wide. Provide full cut-through openings at grade with the street in these cases.

- Provide good drainage at intersection corners so standing water does not accumulate at the crossing area. Place drainage inlets on the uphill side of the crosswalk and outside the crosswalk area.

Figure 13-9: Curb Ramp Details



Source: (23)

13.2.3.10 Transit Facilities

Where transit facilities need to be accommodated within the pedestrian realm, they should primarily occur in the space formed by the combined width of the Edge and Furnishings zones as well as curb extensions. In cases where the combined width of these elements is insufficient to accommodate these facilities, reduce the Through zone width up to the minimums indicated in Table 13-2.

Where the lay-by will compromise the pedestrian realm or cycle facilities, eliminate parking on the frontage lane, and/or narrow the combined width of the Edge and Furnishings zones.

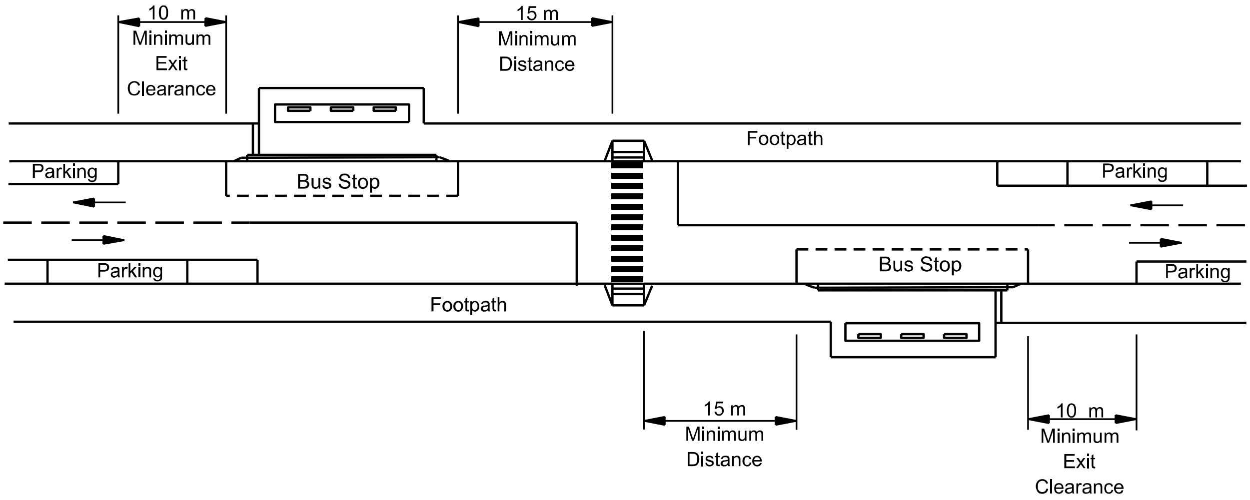
Most transit users walk or cycle to transit stops and in order for a transit system to function effectively, streets should be designed to provide excellent access and high quality facilities for pedestrians and cyclists. Locate transit stops close to activities and uses that will benefit from their proximity. Provide a minimum 2-m obstacle-free clear zone around transit stops to ensure accessibility. Pedestrian crossings should be located before vehicle waiting areas or bus stops to avoid conflicts and delays.

Figure 13-10 through provide guidance for locating bus stops near crosswalks.

13.2.3.11 Construction Materials

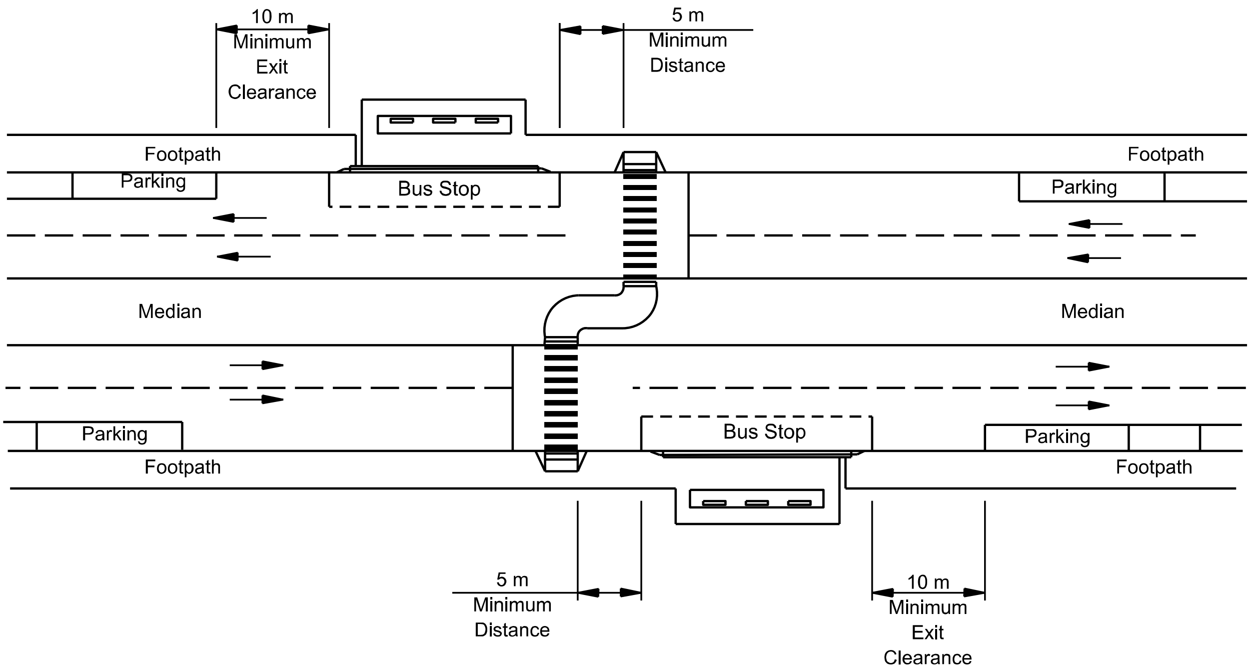
Sidewalks and pathways may be constructed of materials other than concrete with a smooth debris-free surface. The use of asphalt and limestone screenings can contribute to a park-like atmosphere and alleviate the concerns occasionally expressed by some developers and communities that sidewalks and paths are not aesthetically pleasing. Sidewalks and pathways for recreational use need not be elaborate or expensive. Sidewalks and pathways are sometimes constructed away from roadways (e.g. parks, scenic areas), which can be quite desirable facilities for walking.

Figure 13-10: Bus Stops at Mid-Block Crossing (1+1 street)



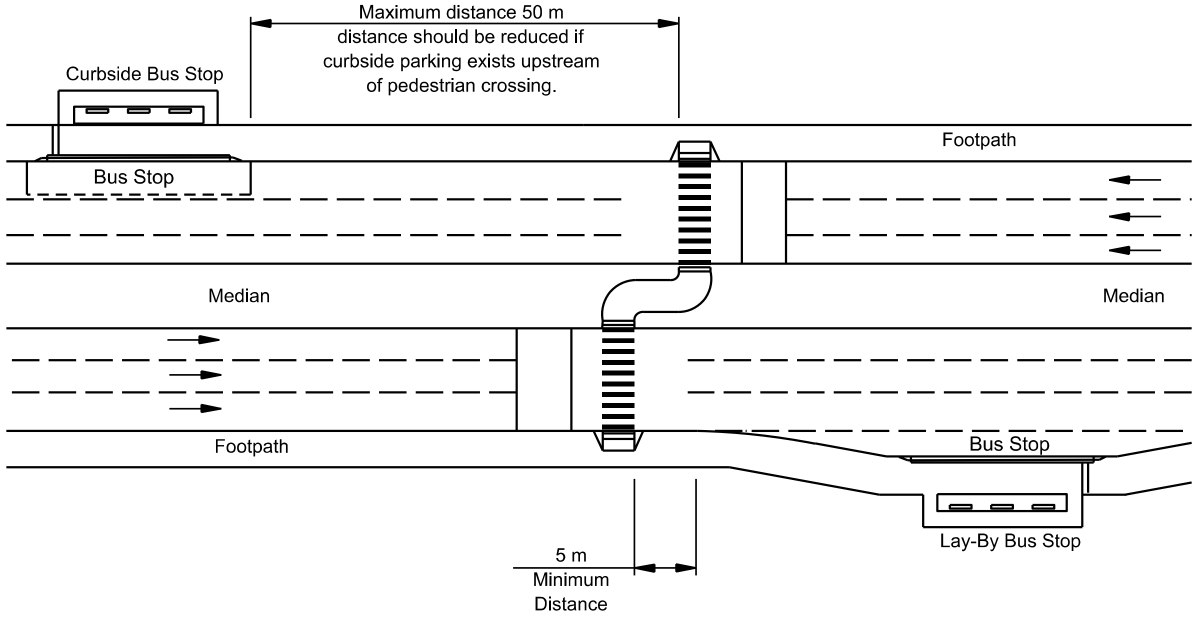
Source: (49)

Figure 13-11: Bus Stop/Lay-By after Mid-Block Crossing (Divided Street)



Source: (49)

Figure 13-12: Bus Stop/Lay-By before Mid-Block Crossing (Divided Street)



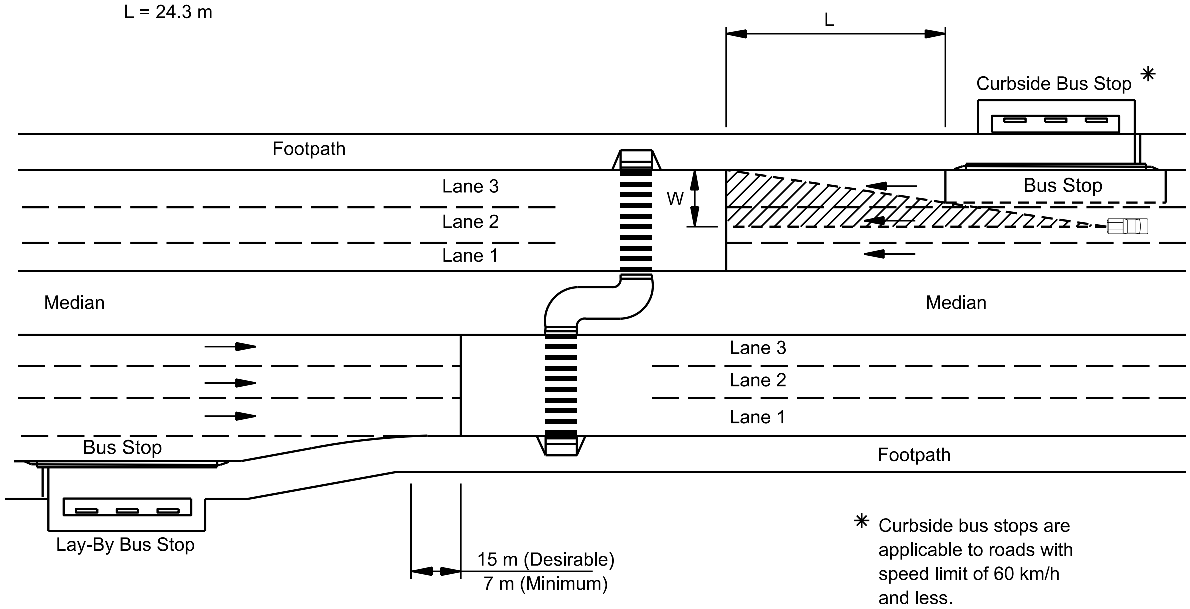
Source: (49)

Figure 13-13: Bus Stop after Mid-Block Crossing (Divided Street)

$$L = \frac{1}{W}\left[\frac{V}{1.2}t + \frac{V^2}{29.3}\right]$$

L = Distance of bus stop to stop line in metres.
V = Approach speed in km/h. Approach speed is subject to the expected speed after the introduction of the pedestrian crossing type.
t = Reaction time in seconds.
W = Distance of curb edge to lane 2 centreline in metres.

Example:
If W = 5 m, V = 40 km/h, t = 2 sec
L = 24.3 m



Source: (49)

13.2.4 Traffic Calming

To manage vehicular speeds and to reduce exposure risk to all users, traffic calming methods may be required. Traffic calming measures are usually physical because of their proven effectiveness in reducing speeds, and collisions (e.g. raised crosswalks, see Section 13.2.3.4).

Traffic calming can be either proactive or reactive. The street cross sections and junction details included in this *Manual* are meant to create streets that have been designed to keep speeds low and crossings short. At locations where pedestrian safety is of concern (e.g. near schools), designers should use traffic calming devices as part of the new street design process. After streets are built, if designers find that traffic speeds or volumes are excessive, or that safety is compromised, traffic calming devices should be retrofitted into the problem streets. In both scenarios, design and implementation are an iterative process that requires monitoring and adjustment. For guidance on traffic calming methods, see the DMT *Abu Dhabi Urban Street Design Manual* (4).

13.3 Cyclists

This section addresses cyclist (bicycle) facilities and provides guidance as to their intended use, location, and width. For additional guidance on cycle facilities, see the Abu Dhabi Urban Planning Council *Urban Street Design Manual* (4), Abu Dhabi *Walking and Cycling Master Plan* (23), and AASHTO *Guide for the Development of Bicycle Facilities* (58).

13.3.1 Definitions

The following terms and definitions apply to Section 13.2.4; see :

1. <u>Cycle Track</u>. A facility reserved for cyclists and separated from motor vehicle traffic.
2. <u>Cycle Lane</u>. A lane within the travelled way reserved for cyclists.

Key principles for cycle lanes are as follows:

- A Cycle lane is a one way cycle facility marked on a road surface.
- Cyclists have exclusive use of the cycle lane.
- Located on local roads with low volume and speed of traffic.
- Corridor width should more than 11 metres with a minimum carriageway width of 6 metres.
- Separation buffer should be used where cycle lanes are located alongside on street parking.
- Where the carriageway is less than the corridor allowance then cyclists should make use of the shared lane proposal shown below.

3. <u>Shared Roadway</u>. Any roadway upon which a separate cycle lane is not designated and which may be legally used by cyclists regardless of whether such facility is specifically designated as a cycle path (e.g., Avenue, Access Lane, frontage road).

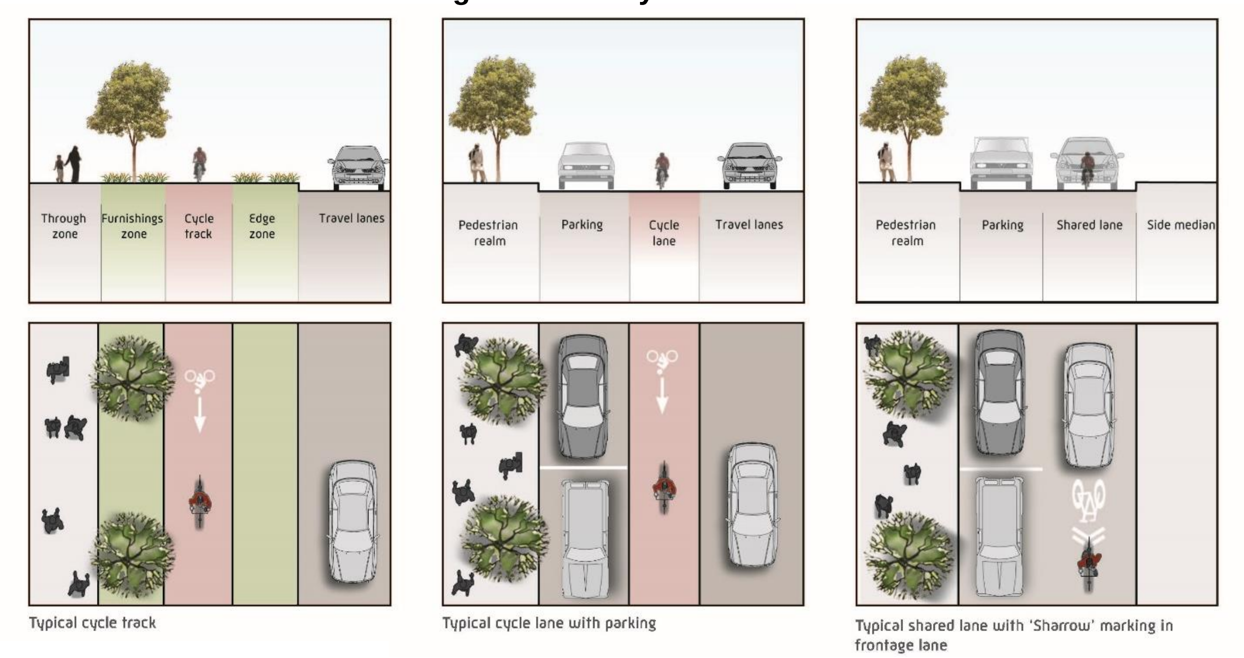
Key principles for shared and mixed traffic facilities are as follows:

- Cyclists share the road with other users, with no demarcation between different modes.
- Located on local roads which are not wide enough to accommodate cycle lanes.
- Generally used on roads that have low traffic volumes and traffic speed (less than 45 kph 85^{th} percentile speed).
- Presence of shared lane is reinforced with additional road markings showing a bicycle and direction of travel.

All new urban streets are required to provide accommodations for cyclists. Selection of the type of a cyclist facility is largely a function of vehicular speeds, traffic volumes, expected cyclist's skills, and available right-of-way width. Facility selection is an iterative process. A higher quality facility will encourage additional ridership, including that of children and older adults. Projected ridership volumes will affect the type and width of cyclist facilities. For guidance on location of cyclist facilities, see the Abu Dhabi *Walking and Cycling Master Plan* (23).

Cyclists can also share space with pedestrians in some low density areas if projected volumes do not warrant a separate facility. Coordination with the client is necessary to confirm the application of this facility.

Figure 13-14: Cycle Facilities



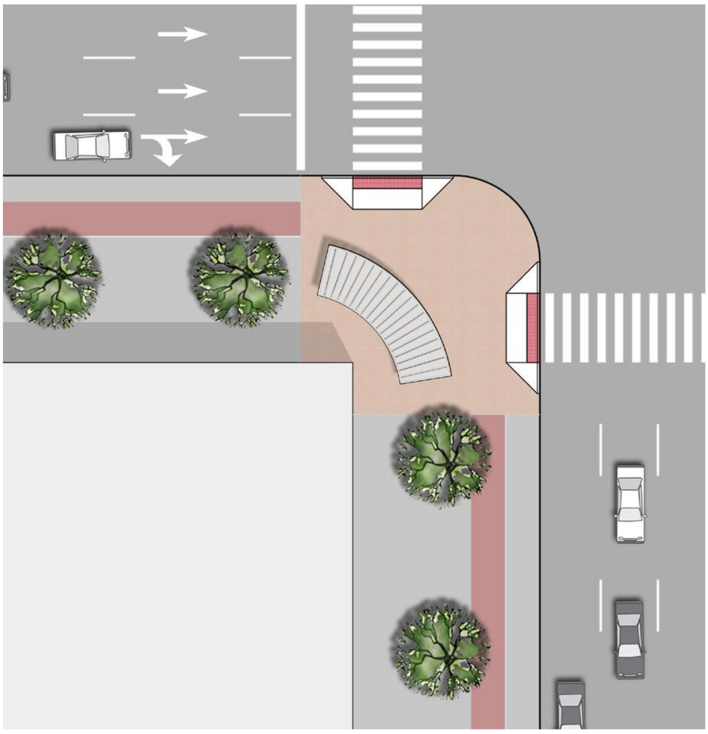
Source: (4)

13.3.2 Cycle Facilities at Intersections

On-road cyclists movements through intersections should be an integral part of a roadway improvement. Specific provisions for cyclists are necessary at both major and minor intersections and at driveways. The Abu Dhabi *Walking and Cycling Master Plan* (23) provides guidance on the treatment of cycle facilities at intersections. In addition, the designer should consider the following:

- Continue cycle tracks at the pedestrian level at raised crosswalks. In other cases, cycle crossings should be at road level.
- Cyclists share space with pedestrians at intersections, in the pedestrian realm and adjacent to crossing facilities; see .
- Mark and colour cycle lanes through the intersection.
- Only use an Advanced Stop Line (ASL) or cycle box under direction from the client. If used, provide a minimum 4.0-m cycle box and surface in the same colour as the cycle lane.
- While cyclist facilities must remain continuous, they may be transitioned from a cycle lane to a cycle track to accommodate changing conditions along the urban street.
- At major intersections, cyclists crossings should be separated from through vehicle traffic.
- Design crossing locations with sufficient space to accommodate cyclists mixing with pedestrians.

Figure 13-15: Shared Waiting Space for Cyclists and Pedestrians at Intersection



Source: (4)

13.3.3 Cycle Parking

The following are general design criteria for cycle parking facilities:

- Locate parking in furnishings zone, out of the through zone or driveways, on curb extensions within 15 m of the main entrance or between buildings.
- Provide long-term cycle parking in convenient, shaded, well-lit, and secure locations.
- Provide directional parking signage if parking facilities are not readily visible to visitors.
- Encourage business to provide cycle lockers to promote cycle commuting.

Cycle racks should be durable and securely anchored. They should be designed so that:

- The cycle frame and at least one wheel can be locked.
- The cycle frame can be supported in at least two places.
- Rack spacing is such that cycles can park without disturbing one another.

13.3.4 Separate Cycle Facilities

13.3.4.1 Purpose

Separate cycle paths are facilities with exclusive right-of-way and with minimal cross flow by motor vehicles. Separate cyclist facilities are sometimes referred to as trails. Care should be taken in using these terms interchangeably. Where separate cycle facilities are called trails, they should meet all design criteria for separate cycle facilities to be designated as such. Users are non-motorised and may include, but are not limited to, cyclists, in-line skaters, roller skaters, wheelchair users (both non-motorised and motorised), and pedestrians, including walkers, runners, people with baby strollers, etc. These facilities are most commonly designed for two-way travel and the guidance herein assumes a two-way facility is planned unless otherwise stated.

Separate cycle facilities can serve a variety of purposes. They can provide users with a shortcut through a residential neighbourhood (e.g. connection between two cul-de-sac streets). Located in a park, they can provide an enjoyable recreational opportunity. Separate cycle facilities can be located along ocean fronts, canals, active railroad and utility rights-of-way, limited access freeways, or within and between parks.

Separate cycle facilities should be thought of as a complementary system of off-road transportation routes for cyclists and others that serve as a necessary extension to the roadway network. Separate cycle facilities should not be used to preclude on-road cycle facilities, but rather to supplement a system of on-road cycle lanes, wide outside lanes, paved shoulders, and cycle routes.

13.3.4.2 Separation Between Cycle Facilities and Roadways

Where two-way separate cycle facilities are located immediately adjacent to a roadway, some operational problems are likely to occur. In some cases, separate cycle facilities along rural roadways for short sections are permissible, given an appropriate level of separation between the two facilities. Some problems with facilities located immediately adjacent to roadways are as follows:

- Unless separated, they require one direction of cycle traffic to ride against motor vehicle traffic, contrary to normal rules of the road.
- When the facility ends, cyclists going against traffic will tend to continue to travel on the wrong side of the street. Likewise, cyclists approaching a separate cycle facility often travel on the wrong side of the street in getting to the facility. Wrong-way travel by cyclists is a major cause of cyclist/automobile crashes and should be discouraged.
- At intersections, motorists entering or crossing the roadway often will not notice cyclists approaching from their right, as they are not expecting contra-flow vehicles. Motorists turning to exit the roadway may likewise fail to notice the cyclist. Even cyclists coming from the left often go unnoticed, especially when sight distances are limited.
- Because of the proximity of motor vehicle traffic to opposing cyclist traffic, barriers are often necessary to keep motor vehicles out of separate cycle facilities and cyclists out of traffic lanes. These barriers can represent an obstruction to cyclists and motorists, can complicate maintenance of the facility, and can cause other problems as well.

Where rural two-way separate cycle facilities are located adjacent to a roadway, wide separation between a separate cycle facility and the adjacent roadway is desirable to demonstrate to both the cyclist and the motorist that the path functions as an independent facility for cyclists and others. Where this is not possible and the distance between the edge of the shoulder and the shared use path is within clear zone, a suitable positive separation by physical barrier or Vehicle Restraint systems (VRS) should be provided. These barriers serve both to prevent path users from making unwanted movements between the path and the road shoulder and to reinforce the concept that the path is an independent facility. Where used, the barrier should be a minimum of 1.1 m high, to prevent cyclists form toppling over it. A barrier between a separate cycle facility and adjacent road should not impair sight distance at intersections, and should be designed to not be a hazard to errant motorists.

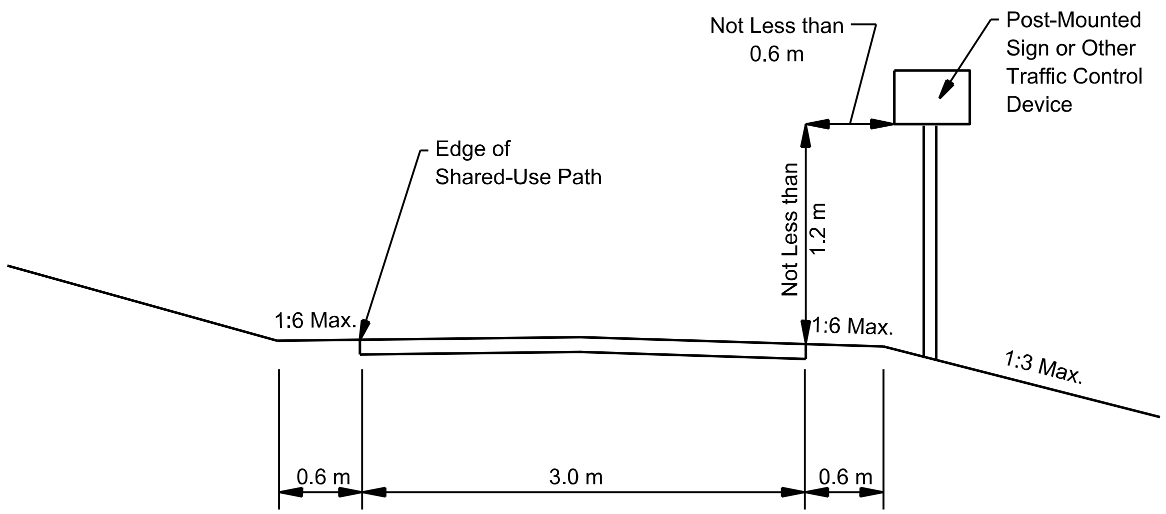
13.3.4.3 Width and Clearance

The paved width and the operating width required for a separate cycle facility are the primary design considerations. The following apply:

1. Pedestrian Realms. Table 13-2 provides widths for cycle facilities within urban pedestrian realms.

2. Rural Cycle Facilities. Figure 13-16 depicts a separate rural cycle facility on a separated right-of-way. The minimum width of a one-directional separate cycle facility is 2.0 m. It should be recognised that one-way facilities often will be used as two-way facilities unless effective measures are taken to ensure one-way operation. Without such enforcement, assume that these facilities will be used as two-way facilities by both pedestrians and cyclists and designed accordingly.

Figure 13-16: Cross Section of Two-Way Shared Use Path on Separated Right-of-Way



Source: (58)

3. Rural Two-Directional Facilities. Under most conditions, the preferred paved width for a two-directional facility is 3.0 m. In rare instances, a reduced width of 2.4 m can be adequate. This reduced width should be used only where the following conditions prevail:

    - cyclist traffic is expected to be low, even on peak days or during peak hours;
    - pedestrian use of the facility is not expected to be more than occasional;
    - there will be good horizontal and vertical alignment providing safe and frequent passing opportunities; and

- during normal maintenance activities, the path will not be subjected to maintenance vehicle loading conditions that would cause pavement edge damage.

4. Shared Use. Under certain conditions, it may be necessary or desirable to increase the width of a separate cycle facility to 3.6 m, or even 4.2 m, due to substantial use by cyclists, joggers, skaters, and pedestrians, use by large maintenance vehicles, and/or steep grades.
5. Side Slopes. A minimum 0.6 m wide graded area with a maximum 1:6 slope should be maintained adjacent to both sides of the facility; however, 1.0 m or more is desirable to provide clearance from trees, poles, walls, fences, guardrails, or other lateral obstructions. Where the path is adjacent to canals, ditches, or slopes steeper than 1:3, a wider separation should be considered. A minimum 1.5 m separation from the edge of path pavement to the top of the slope is desirable. Depending on the height of embankment and condition at the bottom, a physical barrier (e.g. dense shrubbery, railing, chain link fence) may need to be provided.
6. Vertical Clearance. The vertical clearance to obstruction should be a minimum of 2.5 m. However, the vertical clearance may need to be greater to permit passage of maintenance and emergency vehicles. In under crossings and tunnels, a vertical clearance of 3.0 m is desirable.

13.3.4.4 Design Speed

The cyclist's design speed will vary depending upon the user's physical condition and age; type and condition of equipment; purpose and length of trip; the condition, location, and grade of the path; and the type of other users using the path (e.g. walkers). In consideration of the noted factors, the following guidance is offered in selecting the design speed:

1. Level. For paths in relatively flat areas (i.e. grades less than 2 percent), a design speed of 30 km/h is generally considered to be sufficient. Higher speeds can be expected on inclines. In congested areas, lower design speeds are appropriate.
2. Hilly. In hilly areas and where grades are 6 percent or greater, the design speed should be selected based on the cyclist going downhill. In all but the extreme cases, a design speed of 50 km/h is the maximum design speed that should be used.

See the *AASHTO Guide for the Development of Bicycle Facilities* (58) for further guidance on determining cyclist's design speeds.

13.3.4.5 Stopping Sight Distance

Stopping sight distances for cyclists can be determined by using Equation 13.1. Assuming a typical reaction time of 2.5 seconds and friction factor of 0.16, Table 13-4 provides stopping sight distance for typical cyclists on level grades.

$$\mathbf{SSD} = \frac{\mathbf{Vt}}{\mathbf{3.6}} + \frac{\mathbf{V^2}}{\mathbf{254\left(f \pm G\right)}}$$

Equation 13.1: Cycle Stopping Sight Distances

where: SSD = stopping sight distance, m

t = reaction time, sec (assume 2.5 seconds)
V = design speed, km/h
f = friction factor, (assume 0.16)
G = longitudinal grade, decimal (+ for upgrades and – for downgrades)

Table 13-4: Stopping Sight Distance (Cycles — Level Grade)

Design Speed (km/h)	Stopping Sight Distance (m)
20	24
25	33
30	43
35	54
40	67
45	81
50	96

13.3.4.6 Horizontal Alignment

For separate paved cycle paths, the horizontal alignment for cyclists is generally determined based on the cyclist's lean in cornering. A 20 degree is considered to by typical for most bicyclists. Equation 13.2 can be used to determine the minimum radius of curvature based on the amount of lean and design speed. Table 13-5 provides the minimum radii for horizontal curves on paved paths assuming a 20 degree lean angle.

$$R = \frac{V^2}{127\left(\tan\theta\right)}$$

Equation 13.2: Minimum Radius of Curvature Based on Lean Angle

where: R = radius of curve, m
V = design speed, km/h
ϴ = lean angle from vertical, degrees

Source: (58)

Table 13-5: Minimum Radii for 20-Degree Lean Angle

Design Speed (km/h)	Minimum Radius (m)
20	9
25	14
30	19
35	27
40	35
45	44
50	54

A second method for determining the horizontal curvature may be considered using Equation 5.1 assuming a superelevation rate and side friction factors. These methods may be considered on unpaved paths and on paved paths where only cyclists are allowed. See the *AASHTO Guide for the Development of Bicycle Facilities* (58) for further guidance on determining minimum radii using Equation 5.1.

13.3.4.7 Crest Vertical Curves

For separated cycle paths, crest vertical curves can be determined using the equations presented in Section 6.4.1 and using a typical cyclist's eye height of 1.4 m and an object height of 0.0 m.

14 RAILWAY CROSSINGS

14.1 Overview

A road-railroad crossing, like any road intersection, involves either grade separation or at-grade crossing. The geometrics of a road and structure that involves the overcrossing or undercrossing of a railroad are substantially the same as those for a roadway grade separation without ramps. For crossings with structures, the designer is referred to the Abu Dhabi *Road Structures Design Manual* (29).

This chapter provides guidelines for at-grade road-railroad crossings. These guidelines are not all inclusive. Situations not covered by these guidelines should be evaluated using good engineering judgment.

14.2 Geometric Design Elements

14.2.1 General Guidance

The design of a road-railroad grade crossing involves the elements of alignment, profile, sight distance, and cross section. The geometric design may vary with the type of warning device used. Where signs and pavement markings are the only means of warning, the road should cross the railroad at or nearly at right angles. Even when flashing lights or automatic gates are used, avoid using small intersecting angles. Regardless of the type of control, provide a roadway gradient that is flat at and adjacent to the railroad crossing to permit vehicles to stop, when necessary, and then proceed across the tracks without difficulty.

Coordinate with the railroad authority to determine the appropriate warning devices to be used; see Section 14.3. When only passive warning devices (e.g. signs and pavement markings) are used, drivers are warned of the crossing location, but need to determine for themselves whether there are train movements for which they should stop. Where active warning devices (e.g. flashing light signals, automatic gates) are used, the driver is given a positive indication of the presence or the approach of a train at the crossing. For low-volume crossings and where adequate sight distance is not available, provide additional signing.

14.2.2 Sight Distance

Sight distance is a major consideration at crossings without train-activated warning devices. The methodologies for determining sight distances are discussed in Chapter 4 “Sight Distance,” which are also applicable to road-railroad crossing.

As in the case of a road intersection, several events occur at a road-railroad grade intersection without train-activated warning devices. The sight distance events include:

- The driver can observe the approaching train in a sight line that will allow the vehicle to pass through the grade crossing prior to the train’s arrival at the crossing.
- The driver can observe the approaching train in a sight line that will permit the vehicle to be brought to a stop prior to encroaching in the crossing.

These manoeuvres are shown as Case A in Figure 14-1. The sight triangle consists of the two major legs (i.e. the sight distance (d_H) along the roadway and the sight distance (d_T) along the railroad tracks). Table 14-1 provides the sight distances based on the speeds of the vehicle and train. These distances were developed from the following equations:

$$d_H = AV_v t + \frac{BV_v^2}{a} + D + d_e$$

Equation 14.1 Sight Distance along Roadway

$$d_T = \frac{V_T}{V_V}\left[AV_v t + \frac{BV_v^2}{a} + 2D + L + W\right]$$

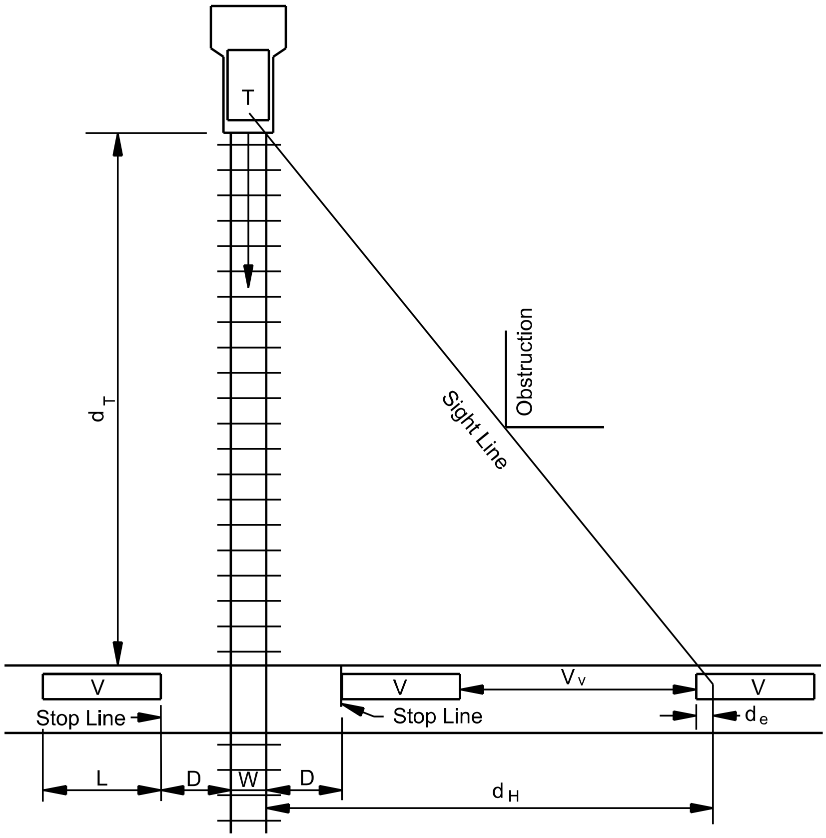
Equation 14.2 Sight Distance along Tracks

where:

A	=	constant = 0.278
B	=	constant = 0.039
d_H	=	sight-distance leg along the road that allows a vehicle proceeding to speed V_v to cross tracks even though a train is observed at a distance d_T from the crossing or to stop the vehicle without encroachment of the crossing area, m
d_T	=	sight-distance leg along the railroad tracks to permit the manoeuvres described as for d_H, m
V_v	=	speed of the vehicle, km/h
V_T	=	speed of the train, km/h
t	=	perception/reaction time, which is assumed to be 2.5 s
a	=	driver deceleration, which is assumed to be 3.4 m/s^2
D	=	distance from the stop line or front of the vehicle to the nearest rail, which is assumed to be 4.5 m
d_e	=	distance from the driver to the front of the vehicle, which is assumed to be 2.4 m
L	=	length of vehicle, which is assumed to be 22.4 m
W	=	distance between outer rails, for a single track, this value is 1.5 m

Note: Adjustments should be made for skewed crossings and roadway grades that are other than flat.

Figure 14-1: Moving Vehicle to Safely Cross or Stop at Railroad Crossing



Source: (1)

Table 14-1: Sight Distance for Crossing a Single Set of Tracks

Train Speed (km/h)	Case B Departure from Stop	Case A Moving Vehicle												
		Vehicle Speed (km/h)												
		10	20	30	40	50	60	70	90	90	100	110	120	130
Distance along railroad from crossing, d_T (m)														
10	48	41	26	21	20	19	19	20	20	21	22	23	23	24
20	96	82	51	43	40	39	39	39	40	42	43	45	47	49
30	143	123	77	64	59	58	58	59	61	63	65	68	70	73
40	191	164	103	85	79	77	77	79	81	84	87	90	94	98
50	239	205	128	107	99	96	97	98	101	105	109	113	117	122
60	287	246	154	128	119	116	116	118	121	126	130	135	141	146
70	334	287	180	150	138	135	135	138	142	147	152	158	164	171
80	382	328	206	171	158	154	155	157	162	167	174	180	188	195
90	430	369	231	192	178	173	174	177	182	188	195	203	211	220
100	478	410	257	214	198	193	193	197	202	209	217	226	235	244
110	526	451	283	235	217	212	212	216	223	230	239	248	258	268
120	573	492	308	256	237	231	232	236	243	251	261	271	281	293
130	621	533	334	278	257	250	251	256	263	272	282	293	305	317
140	669	574	360	299	277	270	270	276	283	293	304	316	328	341
Distance along road from crossing, d_H (m)														
		15	25	38	53	70	90	112	136	162	191	222	255	291

These sight distances where developed assuming a 22.4-m truck crossing a single set of tracks at 90 degrees. Use Equations 14.1, 14.2, and 14.3 to determine the sight distance for other site conditions. *Source*: (1)

After a vehicle has stopped at a railroad crossing, the next manoeuvre is to depart from the stopped position (Case B). The driver should have sufficient sight distance along the tracks to accelerate the vehicle and clear the crossing prior to the arrival of a train, even if the train comes into view just as the vehicle starts. See Figure 14-2. These values are obtained from the equation:

$$\mathbf{d_T = AV_T\left[\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_G} + J\right]}$$

Equation 14.3 Sight Distance along Track (Stopped)

where: d_T = sight distance leg along railroad tracks to permit the manoeuvres described as for d_H, m

A = constant = 0.278

V_T = speed of train, km/h

V_G = maximum speed of starting vehicle, which is assumed to be 2.7 m/s

a_1 = acceleration of starting vehicle, which is assumed to be 0.45 m/s^2

L = length of vehicle, which is assumed to be 22.4 m

D = distance from stop line to nearest rail, which is assumed to be 4.5 m

J = sum of perception and time to start, which is assumed to be 2.0 s

W = distance between outer rails for a single track, this value is 1.5 m

$$d_a = \frac{V_G^2}{2a_1}$$

Where:

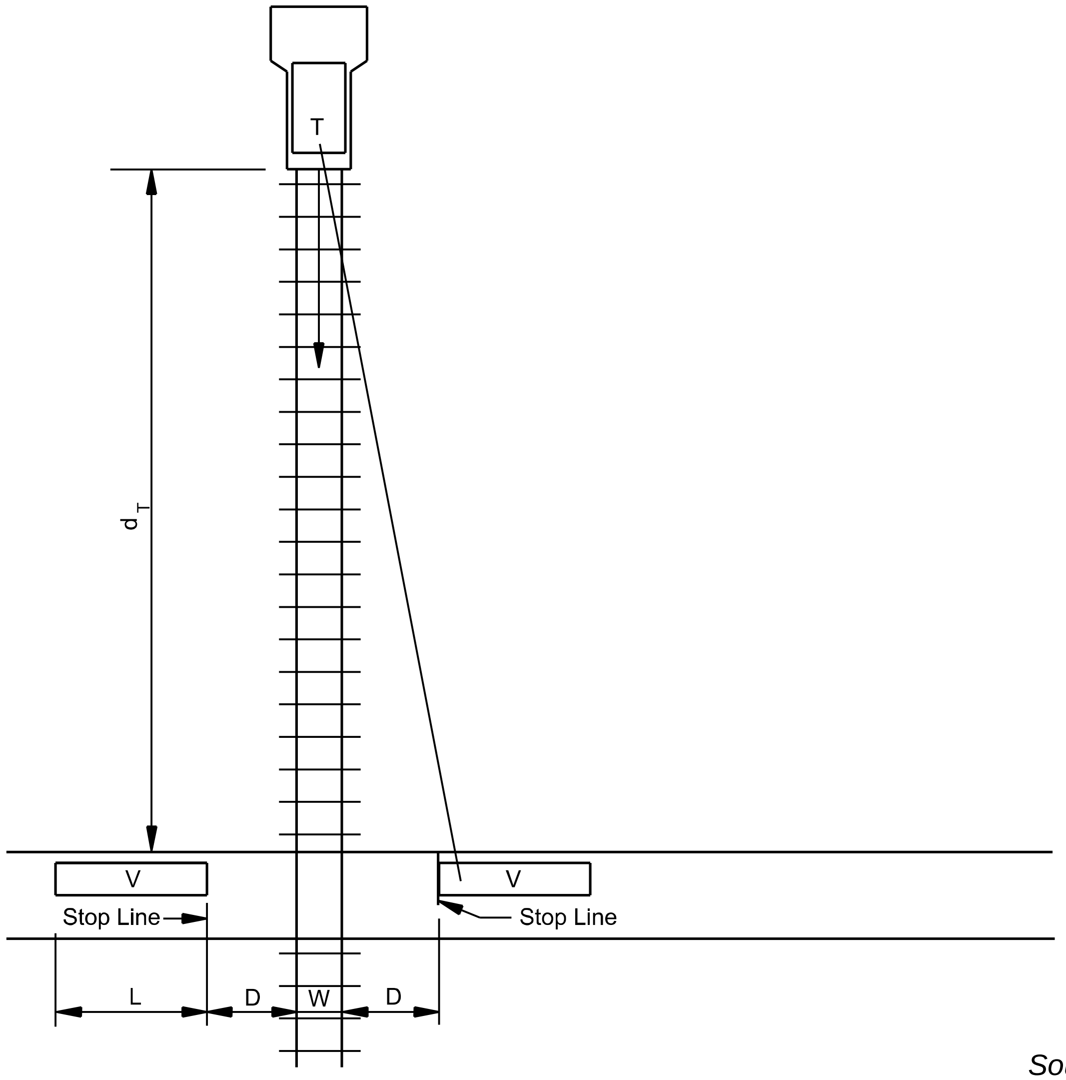
d_a = distance vehicle travels while accelerating to maximum start up speed, m

Note: Adjustments should be made for skewed crossings and roadway grades that are other than flat.

Table 14-1 contains departure sight distances based on the train speed.

Sight distances shown in Table 14-1 are desirable at any railroad grade crossing not controlled by active warning devices. However, they may be difficult to obtain and often are impractical, except in flat, open terrain. In other than flat terrain, it may be appropriate to rely on speed control signs and devices and to predicate sight distance on a reduced vehicle speed of operation. Where sight obstructions are present, install active traffic control devices that will bring all roadway traffic to a stop before crossing the tracks and will warn drivers automatically in time for an approaching train.

Figure 14-2: Departure of Vehicle from Stopped Position to Cross Railroad Track



Source: (1)

The driver of a stopped vehicle at a crossing should see enough of the railroad track to be able to cross it before a train reaches the crossing, even though the train may come into view immediately after the vehicle starts to cross. The length of the railroad track in view on each side of the crossing should be greater than the product of the train speed and the time needed for the stopped vehicle to start and cross the railroad. The sight distance along the railroad track may be determined in the same manner as it is for a stopped vehicle crossing a road, intersection sight distance, which is covered in Chapter 4 "Sight Distance." To cross two tracks from a stopped position, use Equation 14.3 with a proper adjustment for the *W* value.

14.2.3 Horizontal Alignment

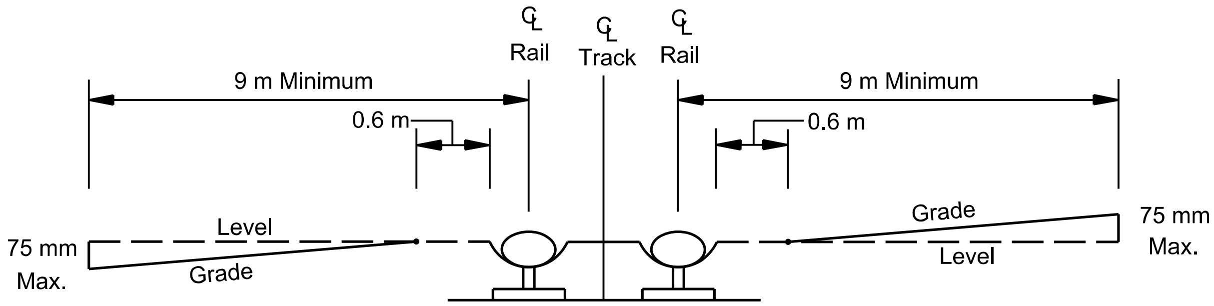
Where practical, the road should intersect the tracks at a right angle with no nearby intersections or driveways. This enhances the driver's view of the crossing and tracks, reduces conflicting vehicular movements from crossroads and driveways, and is preferred for cyclists. To the extent practical, do not locate crossings on either road or railroad curves. Roadway curvature inhibits a driver's view of a crossing ahead, and a driver's attention may be directed toward negotiating the curve rather than looking for a train. Railroad curvature may inhibit a driver's view down the tracks from both a stopped position at the crossing and on the approach to the crossings. Crossings located on both road and railroad curves present maintenance challenges and poor rideability for roadway traffic due to conflicting superelevations.

14.2.4 Vertical Alignment

It is desirable from the standpoint of sight distance, rideability, braking, and acceleration distances that the intersection of road and railroad be made as level as practical. Provide a vertical curve of sufficient length to ensure an adequate view of the crossing.

In some instances, the roadway vertical alignment may not meet acceptable geometrics for a given design speed because of restrictive topography or limitations of right of way. To prevent drivers of low-clearance vehicles from becoming caught on the tracks, ensure the crossing surface is in the same plane as the top of the rails for a distance of 0.6 m outside the rails. The surface of the road should also not be more than 75 mm higher or lower than the top of nearest rail at a point 9 m from the tracks; see Figure 14-3. Use appropriate vertical curves used to traverse from the road grade to a level plane at the elevation of the rails. Rails that are superelevated or a roadway approach section that is not level will require a site analysis to determine the applicable rail clearances.

Figure 14-3: Railroad-Road Grade Crossings



Source: (1)

14.2.5 Surface Design

The roadway surface should be constructed for a suitable length with all-weather surfacing. Carry the current or proposed cross section of the approach roadway through the crossing. The crossing surface itself should have a riding quality equivalent to that of the approach roadway. If the crossing surface is in poor condition, the driver's attention may be devoted to choosing the smoothest path over the crossing. This effort may well reduce the attention given to observance of the warning devices or even the approaching train.

14.2.6 Intersections Design Near Railroad Crossings

Where roads that are parallel with main tracks intersect roads that cross the main tracks, ensure there is sufficient distance between the tracks and the roadway intersection to enable vehicular traffic in all directions to move expeditiously. The designer must ensure vehicles are not required to stop or store on the railroad tracks near an intersection. This applies to both signal- or stop-controlled intersections. Review the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22), Abu Dhabi *Traffic Signals and Electronic Warning and Systems Manual* (45) or contact the client for guidance on the design of intersections near railroad crossings.

14.3 Traffic Control Devices

Traffic control devices for railroad-road grade crossings consist primarily of signs, pavement markings, flashing light signals, automatic gates, and traffic signals for the light rail transit. Criteria for design, placement, instalment, and operation of these devices are covered in the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22), as well as the use of various passive warning devices. Some of the considerations for evaluating the need for active warning devices at a grade crossing include roadway type, vehicular traffic volume, railroad traffic volume, maximum speed of the trains, design speed of vehicular traffic, volume of pedestrian traffic, crash history, sight distance, and geometrics of the crossing.

Motorist comprehension and compliance with each of these devices is mainly a function of education and enforcement. The traffic engineer should make full use of the various traffic control devices (including pavement markings and signs) as prescribed in the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22) to convey a clear, concise, and easily understood message to the driver, which should be facilitated with education and enforcement.

14.4 Light Rail Transit Track in Mixed Traffic

14.4.1 Introduction

While it is highly desirable that light rail transit (LRT) lines have an exclusive or semi-exclusive guide-way, see Figure 14-4, there frequently are circumstances where the only practical or affordable alignment is to place the tracks within a public roadway in a mixed traffic environment, see Figure 14-5.

Where these rail cars operate in general traffic lanes, they are mixed with steerable, rubber-tired vehicles and should, to the greatest extent feasible, flow with traffic. They should not make any unusual or unexpected lateral movements, but rather function like the rubber tired vehicles also using the street. However, there is a fundamental difference between rail cars and motor vehicles. Rail car operators can control the acceleration, deceleration, and speed of their vehicle to match the pace of traffic, but they cannot steer. Vehicle guidance is done by the tracks. This fundamental principle, which is well understood by the track designers, must be considered by roadway designers and traffic engineers.

Track work within streets creates significant discontinuities in the pavement surface by replacing longitudinal segments of conventional paving materials with steel rails plus adjacent open flange ways. To the maximum degree practical, these steel surfaces and openings should not create hazards to other users of the street, including pedestrians and rubber-tired vehicles of all types.

Fitting tracks into locations where the street geometry imposes restrictions is challenging. Designers of the track, roadway, overhead contact system, and light rail vehicle all need to work closely with each other as well as with the project staff preparing the transit operations plan.

Figure 14-4: Proposed Abu Dhabi Tram Station

Figure 14-5: Tram within the Travelled Way

These issues, which meld roadway, traffic, and track engineering, need to be resolved as early as possible in the design process. The designers in each discipline should work in concert as the conceptual design matures.

14.4.2 Track Position Within Lanes

There are a number of factors that affect where the tracks might be placed. These include the actual lane width available, light rail vehicle clearance diagram, presence or absence of station stops, presence of adjoining parking lanes, multiple tracks, oncoming traffic, nearby structures and signs, etc. Because there are no standard criteria for streetcar or LRT vehicle widths, lengths, and suspension and track designs, each system must be designed on a case-by-case basis considering all of the above listed factors. Occasionally, underground utility features that cannot be relocated may also constrain the position of the tracks.

14.4.2.1 Vehicle Clearances

A clearance diagram must be determined considering both the vehicle's static outline and its dynamic outline. The static outline is the shape of the car at rest. The dynamic outline includes the allowable movement in the suspension system, end overhang, and mid-ordinate overhang. The manufacturer develops the dynamic outline for each type of transit vehicle. To establish clearances along the right-of-way, a vehicle dynamic clearance envelope must also be developed. Using the vehicle dynamic outline along with the associated track components, track tolerances, wear limits of the components, and a clearance zone with a safety factor of 50 mm, the dynamic vehicle clearance envelope can be established.

Where facility clearance restrictions exist, the track designer should coordinate with the vehicle and structural designers to ensure that adequate car clearance is provided. Vehicle dynamics are governed by the cars suspension system(s) and, therefore, indirectly by numerous factors of track and vehicle interaction. For multiple-track situations, multiple clearance envelopes must be considered. Overlapping must be avoided. The resulting requirements will dictate minimum track centres and clearances for tangent and curved track, including tolerances and safety factors.

14.4.2.2 Transverse Position Within Lanes

Because the coefficient of friction between rubber and steel is lower than that of rubber on paving, especially when the roadway is wet, it is generally better for rubber-tired vehicles to travel with all tires on paving, rather than on steel rails. Offsetting the centrelines of the track and traffic lane facilitates this goal. The direction of the offset is immaterial; the intended effect is achieved either way. However, site-specific conditions (e.g. all stations are on the right side of the track) might indicate a preference for a particular juxtaposition of the two centrelines.

Arguably, the two most important defining elements of track work for light rail systems are the construction of track in streets and the interface between the wheel of the light rail vehicles and the rails. Track in streets requires special consideration, especially with regard to the control of stray electrical current that could cause corrosion. These embedded tracks also need to provide a flangeway that is large enough for the wheels but does not pose a hazard to other users of the street.

While light rail may need to share right-of-way with pedestrians and vehicles, the designer should create an exclusive right-of-way for light rail tracks, wherever practical. This will make operations more reliable and maintenance less expensive.

14.4.2.3 Adjacent Parking Lanes

Situations in which a parking or curb loading lane is positioned alongside of the shared lane used by LRVs must be carefully considered. Typically, these lanes are no wider than 2.5 m, a dimension sufficient for an ordinary automobile, but not for a wide delivery truck. These trucks can have body widths of 2.59 m, plus mirrors, and will thereby frequently overhang the line, even if the truck driver is diligent about getting the tires close to the curb.

In these situations, if the position of the adjoining LRT track is biased toward the parking lane, it is likely that badly parked delivery trucks will encroach into the dynamic envelope of the LRV and perhaps even its static outline. In the latter case, if the truck driver cannot be found quickly, light rail service might be blocked for an extended period until the offending motor vehicle can be moved. Because the LRV may be stopped in a general traffic lane, ordinary motor vehicle traffic could be impacted as well. Such situations may require that the parking lane width be larger than normal or that the track is biased away from the parking lane, or both.

15 INTELLIGENT TRANSPORT SYSTEMS

15.1 Overview

Intelligent Transport Systems (ITS) is the application of advanced technology to solve transportation problems. ITS supports the movement of people, goods, and services. ITS improves transportation safety and mobility and enhances productivity by using advanced information and communications technologies.

ITS encompasses a broad range of wireless and wire-line communications-based information and electronic technologies. When integrated into the transportation system's infrastructure, and in vehicles themselves, these technologies relieve congestion, improve safety, and enhance Abu Dhabi productivity. The following sections discuss the common ITS issues with respect to the road designer. For detailed guidance on these technologies, see the Abu Dhabi *Traffic Signals and Electronic Warning and Systems Manual* (45).

All ITS structure, systems and sub systems infrastructure that are along the road side and within clear zone need a suitable physical barrier or Vehicle Restraint systems (VRS).

15.2 ITS Design

The purpose of ITS design is to bridge the information gap between real time conditions and user information. ITS design includes ramp metering and data station design and installations and retrofits for vehicle detection stations (VDS), traffic data collection, closed circuit television (CCTV), road weather monitoring, help phone systems, variable message signs (VMS), ramp metering systems, weigh-in-motion systems, automatic number plate recognition systems, and tunnel systems. ITS design assumes that the designer has some working knowledge of traffic signal design and illumination electrical work to aid in the ability to do ITS design work.

15.3 Systems Engineering

Systems engineering is an interdisciplinary approach and means to enable the realization of successful systems. It focuses on defining customer needs and required functionality early in the development cycle, documenting requirements, and then proceeding with design synthesis and system validation while considering the complete problem:

- operations,
- cost and schedule,
- performance,
- training and support,
- test,
- manufacturing, and
- disposal.

Systems engineering integrates all of the disciplines and specialty groups into a team effort forming a structured development process that proceeds from concept to production to operation. Systems engineering considers both the business and the technical needs of all customers with the goal of providing a quality product that meets the user needs.

15.4 Vehicle Detection Stations

15.4.1 General

The control of traffic relates to the movement of vehicles and pedestrians. Because the volume of these movements generally varies at different times of the day, it is desirable to be able to detect approaching movements by placing one or more devices in the path of approaching vehicles or at a convenient location for the use of pedestrians.

Most advanced management systems and technologies in the ITS field rely on real-time traffic data, which reflects current conditions of traffic network. Traffic detection is a critical part in many advanced traffic systems (e.g. responsive ramp metering control, freeway incident detection).

Ramp metering control is the most common technology for reducing freeway congestion. The system measures freeway mainline capacity and traffic flow, and controls the rate at which vehicles enter the freeway mainline. Many studies show that ramp metering increases freeway efficiency, and reduce crashes and recurring congestion.

In freeway incident management systems, detectors generally are used to detect two types of congestion: recurring and nonrecurring. Recurring congestion is predictable at specific locations and times. Nonrecurring congestion is caused by random, temporary incidents (e.g. crashes and other unpredictable events).

Traffic detector technologies are continuously incorporated into new ITS application fields. For example, a portable intelligent transportation system provides traveller information at specific sites to improve safety and operation in work zones. A computerized control system integrates detector (speed sensor) and traveller information dissemination technologies. The control system automatically determines appropriate responses according to current traffic conditions.

Traffic detection systems play important roles not only in traditional transportation management, but also in advanced transportation management systems. Traffic detection systems provide data to meet different needs in transportation fields.

15.4.2 Types of Detection

The different types of vehicle detectors available include, but are not limited to the following types:

1. Intrusive Detection (In-Roadway).

    - Inductive loop detects a change in resonant frequency by the introduction of a metal in the magnetic field of the detection zone.
    - Magnetic/magnetometer detects moving ferrous metal objects.
    - Microloop detects a change by moving metal in the earth's magnetic field. A small inductive loop is placed on top of a magnetometer.

2. Non-Intrusive Detection (Above Roadway or Sidefire).

   - Photo electric/infrared detects a break in a beam of light.
   - Radar/microwave detects moving objects by sending and receiving electronic pulses.
   - Ultrasonic detects sound with a microphone.
   - Video detects a change in a video pixel range.

Non-intrusive detector technologies include active and passive infrared, microwave radar, ultrasonic, passive acoustic, and video image processing. Active infrared, microwave radar, and ultrasonic are active detectors that transmit wave energy toward a target and measure the reflected wave. Passive infrared, passive acoustic, and video image processing are passive detectors that measure the energy emitted by a target or the image of the detection zone.

Some detectors record vehicles whether they are stopped or in motion. Others require that the vehicle be moving at a speed of at least 3 km/h to 5 km/h.

Normal loop or magnetic detectors will operate in either the pulse mode or presence mode. The magnetic detector produces a short output pulse when detection occurs, no matter how long the vehicle remains in the detection area. The normal loop is intended to produce a detector output for as long as a vehicle is in the field of detection.

Another type of detection is the speed analysis system. This system is a hardware assembly composed of two loop detectors and auxiliary logic. The two loops are installed in the same lane a precise distance apart. A vehicle passing over the loops produces two actuations. The time interval between the first and the second actuation is measured to determine vehicle speed.

15.5 Traffic Data Collection

To collect vehicular, cyclists, and pedestrian movements on individual road segments, traffic data is collected by either an automated traffic counter or a traffic field technician. There are two different techniques for measuring the volumes for road segments. One technique is called continuous count data collection method. This is where sensors are permanently embedded into a road, and traffic data is measured all 365 days.

Although providing the most accurate data collection, installing and maintaining continuous count stations is costly. An agency is only able to monitor a very small percentage of the roadway using this method. Most data is generated using short-term data collection methods sometimes known as the coverage count data collection method. Traffic is collected with portable sensors that are attached to the road and record traffic data typically for 2 to 14 days. These are typically pneumatic road tubes, although other more expensive technologies (e.g. radar, laser, sonar) exist.

15.6 Closed Circuit Television Systems

Closed circuit television (CCTV) refers to a video or still picture camera system used to collect images and relay images to a central monitoring location, and project images onto a video monitor, television screen, internet display, or other monitoring equipment.

CCTV cameras are a key part of traffic management systems. The primary benefit of CCTV is the ability to provide visual information required to make informed decisions. CCTV cameras are used for roadway surveillance, verification of incidents detected by other means (e.g. cellular calls, speed detectors), and for assistance in determining appropriate responses to an unplanned event or incident.

Beyond these tasks, cameras can be used for:

- monitoring traffic movements on the mainline and ramps,
- variable message sign verification,
- verification of stranded motorists and incidents, and
- observing localised weather and other hazardous conditions.

Cameras also offer an attractive substitute to loop detectors for traffic data collection because they can easily be deployed unobtrusively on the side of a highway and can be used for other monitoring applications. Video provides a means for visual verification of results and is an active research field with the promise of added higher level analyses. Besides providing traffic measurements equivalent to loop detectors, using video to track vehicles in a scene reveals added information difficult to obtain using loop detectors such as accurate origin destination maps and travel time, vehicle type classification, and irregular path and incident detection. Using video sensors allows collection of both loop detector type traffic measurements as well as precise track based analysis not possible with loop detectors.

15.7 Road Weather Monitoring Systems

A road weather monitoring system (RWMS) is a combination of technologies that collects, transmits, and disseminates weather and road condition information. The component of an RWMS that collects weather data is the environmental sensor station (ESS). An ESS is a fixed roadway location with one or more sensors measuring atmospheric, surface (e.g. pavement and soil), and/or hydrologic (e.g. water level) conditions including:

- Atmospheric sensors — air temperature, barometric pressure, relative humidity, wind speed and direction, precipitation type and rate, visibility distance.

- Surface sensors — pavement temperature and condition (dry, wet, chemical concentration), subsurface temperature.

- Hydrologic sensors (stream, river, and tide levels).

Data collected from environmental sensors in the field are stored on-site in a remote processing unit (RPU) located in a cabinet. In addition to the RPU, cabinets typically house power supply and battery back-up devices. The RPU transmits environmental data to a central location via a communication system. Central RWMS hardware and software collect field data from numerous ESS, process data to support various operational applications, and display or disseminate road weather data in a format that can be easily interpreted by a user. Environmental data may be integrated into automated motorist warning systems and transmitted to traffic management centres, emergency operations centres, and maintenance facilities for decision support. This information may also be used to enhance forecasts and supplement mesoscale environmental monitoring networks.

Transportation managers use environmental data to implement three types of road weather management strategies — advisory, control, and treatment. Advisory strategies provide information on prevailing and predicted conditions to both transportation managers and motorists. Control strategies alter the state of roadway devices to permit or restrict traffic flow and regulate roadway capacity. Treatment strategies supply resources to roadways to minimise or eliminate weather impacts. Many treatment strategies involve coordination of traffic, maintenance, and emergency management agencies. Maintenance managers use road weather information to assess the nature and magnitude of threats, make staffing decisions, plan treatment strategies, minimise costs (e.g. labour, equipment, materials), and assess the effectiveness of treatment activities. Traffic managers may alter ramp metering rates, modify incident detection algorithms, vary speed limits, restrict access to designated routes, lanes or vehicle types (e.g. tractor-trailers), and disseminate road weather information to motorists to influence their travel decisions. Some traffic management centres integrate weather data with traffic monitoring and control software. Emergency managers may employ decision support systems that integrate weather observations and forecasts with population data, topographic data, and road network and traffic data. When faced with flooding, sand storms, or wild fires, emergency managers may use this data to evacuate vulnerable residents, close threatened roadways and bridges, and disseminate information to the public.

15.8 Help Phone Systems

The modern roadside help phone is a high quality, state-of-the-art cellular transmitter that provides the communication link to a mobile telephone network (help line). An electronic interface provides all the functions of a telephone. The unit can be remotely programmed to call a required number at the push of a button.

The enclosure is designed for severe weather conditions, and heavy-duty hardware makes the enclosure vandal resistant. A pushbutton is located in or on the face of the enclosure. A pre-programmed number is called once the emergency push button is pressed. The roadside help phone can be made to call two phone numbers, if required.

Power is typically supplied by long-life maintenance free batteries, which are recharged by a solar panel or an AC main source. The power supply via solar panel can provide a complete stand-alone unit. The system is designed for 15 minutes active and 60 minutes standby mode per 24 hours for a period of up to one week without solar panel re-charging. It uses as little as 60 milliamps current when idle. The help phone can place a diagnostic call to the remote service centre before battery power is too low for further reliable operation.

15.9 Variable Message Signs

A variable message sign (VMS) is an electronic traffic sign often used on roadways to give travellers information about special events. VMS signs warn of traffic congestion, crashes, incidents, roadwork zones, or speed limits on a specific roadway segment. In urban areas, VMS are used within parking guidance and information systems to guide drivers to available parking spaces. They may also ask drivers to take alternative routes, limit travel speed, warn of duration and location of the incidents, or inform of the traffic conditions.

A complete message on a panel generally includes a problem statement indicating incident, roadwork, stalled vehicle etc.; a location statement indicating where the incident is located; an effect statement indicating lane closure, delay, etc.; and an action statement giving suggestions what to do to address traffic conditions ahead.

15.10 Ramp Metering Systems

In simple terms, ramp metering is the use of traffic signals at freeway on-ramps to manage the rate of automobiles entering the freeway. Ramp metering systems have been successful in decreasing traffic and improving driver safety. There is one negative aspect that comes with installing ramp metering systems — the cost to the public. Although ramp-metering systems have numerous benefits, the designer cannot overlook their cost. There is a potential for higher levels of emissions, traffic congestion to other roadways, and fuel consumption.

Basically, ramp-metering systems work by allowing one driver to pass through the green signal one at a time creating a 5 to 15 second delay between cars. This helps merge traffic together and decrease the number of potential crashes that may occur during merging traffic. This method has been successful in causing fewer crashes than if a ramp metering system was not in place.

Ramp metering systems slow down vehicular speeds so that the traffic merges at a steady pace. The fewer number of vehicles that enter the roadway at a time will improve safety, but also may cause travel time delays. Ramp metering systems also run at designated times during the day when the traffic is most congested.

Although ramp-metering systems may be an inconvenience to some drivers, ramp-metering systems save lives. Countless studies have shown the decrease in road and on-ramp crashes with the use of ramp metering systems.

Ramp metering is extremely effective in reducing congestion and increasing safety, particularly for ramps within major roadways. By monitoring the traffic flow and associated rate of change, ramp-metering data are used to alleviate ramp congestion and to provide insight to traffic issues between associated roadways. Ramp metering results can be used to direct traffic control systems (e.g. variable message signs, variable speed limit signs).

15.11 Weigh-In-Motion Systems

Roadway weigh-in-motion (WIM) systems are capable of estimating the gross vehicle weight of a vehicle as well as the portion of this weight that is carried by each wheel assembly (half-axle with one or more tires), axle and axle group on the vehicle. Both planners and designers use WIM data.

WIM systems increase the capacity of weigh stations and are often used where heavy truck traffic volumes cannot otherwise be accommodated. WIM systems provide planners and designers with time and date of traffic volume, speed, vehicle classification based on number and spacing of axles, and the equivalent single axle loading that heavy vehicles place on pavements and bridges. The heavy truck axle load data are used by motor vehicle enforcement officers to plan enforcement activities. Software is frequently provided by the manufacturers to aid in system calibration and data analysis.

The accuracy of WIM systems is a function of four principal factors:

- vehicle dynamics;
- pavement integrity, composition, and design;

- variance inherent in the WIM system; and
- calibration.

Vehicle dynamics are dependent on road surface roughness, type of vehicle suspension, vehicle dynamic balance, vehicle weight, vehicle speed, driver manoeuvring, etc. Although WIM systems are generally installed in good pavement, unexpected deterioration or structural anomalies sometimes occur. For instance, WIM measurements worsen when asphalt pavements soften in hot weather and long concrete sections rock along a central axis when a heavy truck passes over the end of the section. The inherent variance of the WIM system is a function of the technology used in the system to measure axle weight.

15.12 Automatic Number Plate Recognition Systems

Automatic number plate recognition (ANPR) is a mass surveillance method that uses optical character recognition on images to read vehicle registration plates. They can use existing closed-circuit television or road-rule enforcement cameras, or ones specifically designed for the task. They may be used by police forces and as a method of electronic toll collection on pay-per-use roads and cataloguing the movements of traffic or individuals.

ANPR can be used to store the images captured by the cameras and the text from the license plate, with some configurable to store a photograph of the driver. Systems commonly use infrared lighting to allow the camera to take the picture at any time of the day. Concerns about these systems have cantered on privacy fears, misidentification, high error rates, and increased government spending.

15.13 Tunnel Management Systems

A tunnel management system (TMS) provides important factors such as traveller real-time information, vehicle guidance, over-height vehicle detection, lane control, incident detection, speed enforcement, and predefined emergency procedures. Highly advanced components are integrated in the system, which are centrally controlled and operated via a graphical user interface. The system is typically monitored and controlled from the control room either on-site or remotely.

At the approach of the tunnel, variable message signs display lane speed limits, lane directions, and a tunnel status. Warning signs advise motorists if the tunnel is open or closed. As an intelligent system, lane speed limits (displayed) are dependent on vehicle volume inside the tunnel and vary when traffic is directed to an alternative route in case of tunnel closure. Lane control directional signs and tunnel status warning signs give drivers an advance indication of the tunnel’s condition even before entering its approach, and facilitate lane merging as necessary.

Additional crash prevention features are integrated into the system. Over speeding vehicles approaching the tunnel are detected by cameras and are warned with flashing signals. In advance of the tunnel approach, an over-height vehicle detection system is installed to detect vehicles exceeding permitted height and directs them to alternative routes.

Inside the tunnel, lanes are illuminated with light emitting devices (LEDs) that are mounted on the road. In case of an emergency, motorists inside the tunnel are directed to the nearest escape with illuminated LED signs. Emergency roadside telephones are typically available inside and outside of the tunnel, with default calling to the tunnel’s control centre and emergency services.

A camera system with automatic incident detection capability can be installed in the tunnel. This camera could trigger an alarm when it detects slow traffic movement, traffic flow in the wrong direction, close vehicle gaps, high speed differential, or even unidentified objects on the road. With its continuous recording, the operator could be prompted with pre and post video recordings, which trigger the alarm that will relieve the operator from constantly monitoring the video wall. This system guarantees that the operator gains more information on the alarm cause, and could help validate an emergency case, and eventually activate necessary predefined emergency settings for the entire system.

In the event of a crash inside the tunnel, the incident detection cameras trigger an alarm, which the operator can validate. If the incident is severe and requires closure of the tunnel, the operator can select a predefined configuration for the system to display all the necessary information on the variable message signs at the approach to the tunnel to direct all vehicles to an alternative route. The tunnel can then be closed to traffic until the incident is resolved to reduce further complications.

15.14 Over-Height Vehicle Detection System

In general, over-height vehicle detections systems are deployed to warn drivers if their vehicle exceeds the maximum height for the DMToming infrastructure, whether that be a tunnel entrance, low bridge, sign gantry, parking structures, airport overhangs/walkways, construction sites, etc. Typically these systems:

- detects over-height vehicles and warns drivers of an impending problem.
- alerts and directs the driver via warning signs and warning sounders to take corrective action.
- provides secondary warning beyond existing signage in the interest of public safety.
- reduces exposure to costs associated with incidents or crashes.
- proven to minimise or eliminate the occurrence of crashes and incidents caused by over-height vehicles.

Any over-height detection system must have the ability to integrate with electronic warning signs that will illuminate upon detection from over-height detectors. The minimum requirements for a warning sign shall consist of a mandatory message directing over-height vehicles to the next exit or nearest refuge zone supplemented by flashing warning beacons.

16 ROADSIDE AMENITY

16.1 Overview

The roadside design has a significant impact on the serviceability, safety, functionality, and visual quality of the facility. This chapter presents a discussion on several roadside design elements from the perspective of the roadway designer. These include:

- visual amenity and aesthetics,
- landscaping,
- fencing,
- rest areas, and
- off-street parking.

The proper planning, location, and design for these elements will significantly enhance the roadway's value to the travelling public.

16.2 Visual Amenity and Aesthetics

16.2.1 Introduction

This section discusses the identification of the potential visual impacts of a roadway project and assessing the nature of these effects. Within the framework of this approach, the choice of the visual and aesthetic elements of a project design should be tailored to the specific site. See the Abu Dhabi *Environmental Assessment for Road Projects Manual* (20).

The visual environment and aesthetics of an area are closely linked to the land use and development. Over the last few decades, Abu Dhabi has experienced large urban growth. The number of buildings in the Emirate has more than doubled from 1985 to 2005. Small towns have been transformed from mud-walked communities into fully modern urban centres characterised by skyscrapers in the commercial cities, multi-story residential buildings, large shopping malls, wide boulevards, an extensive network of highways, and sprawling new suburbs.

Currently, the Emirate does not have established standards that specifically address visual and aesthetic requirements during project phases. This topic is not included in the Technical Guidance documents for any of the environmental assessments. Regardless, the visual and aesthetic environment should be considered when preparing an environmental assessment document and evaluating the impacts of all project alternatives. Coordination with the Abu Dhabi Urban Planning Council is important to ensure compliance with local plans and Plan 2030.

16.2.2 Importance

The public nature and visual importance of roadways require that visual impacts, positive and negative, be adequately assessed and considered when a roadway project is developed. Community satisfaction with the project may also be strongly influenced by its visual effects.

Project visual impacts are seen in both the view from the road and the view of the road. The importance of the first has long been recognised. Researchers have also shown that the view from the road is the basis for much of what we know about the everyday environment and for the mental image of the local area.

Systematic consideration of the view of the road is more recent. Particularly in urban areas, there may be many residents along the right-of-way of a proposed project. If existing views are very high in quality or are valued by large numbers of people, the visual costs borne by roadway neighbours could outweigh the visual benefits accrued by roadway users. In such cases, projects must be carefully planned to ensure that pleasing vistas for travellers are not developed at the expense of views from surrounding areas.

The designer must carefully consider existing visual resources that are high in quality and that enhance the built environment by good project planning and design. A systematic approach to visual impact assessment will help achieve attractive roadway projects that are appropriate to their viewers and visual settings.

16.2.3 Objectives

The Abu Dhabi Emirate is committed to aesthetic quality in roadway design. The objectives are to:

- develop a basic understanding of the principles of aesthetics and how they apply to roadway planning, location, and design;
- as part of the environmental assessment process, identify and evaluate location and design alternatives that minimise or eliminate adverse impacts on existing views and that enhance the visual benefits of roadway projects; and
- document the visual impacts for the environment document.

16.2.4 Aesthetics and Visual Impact Assessment

16.2.4.1 General

This section discusses the principles of aesthetics that apply to visual impact assessment. It places aesthetics and visual experience in the context of the environmental assessment, discusses how to identify the visual environment of a project, and examines the viewers and visual resources in that environment, including the roadway itself. Therefore, the designer must understand what aesthetics means within the context of the environmental assessment before adequately evaluating visual impacts.

16.2.4.2 Aesthetics and the Environment

Aesthetics is the science or philosophy concerned with the quality of visual experience. The designer cannot meaningfully assess project impacts on visual experience unless one considers both the stimulus and the response aspects of that experience.

The word quality can refer simply to an attribute or characteristic of a subject. However, quality also can mean excellence or superiority in kind. The statement of need for the project will recognise the critical importance of restoring and maintaining environmental quality. This implies that aesthetic assessments must not only describe the visual attributes of projects, but must also evaluate their effects on the relative excellence of the visual experience.

16.2.4.3 Three Levels of Project Aesthetics

The emphasis on the quality of the overall environment has expanded the context in which the designer must assess project aesthetics. Traditionally, visual design theory has followed the lead of the fine arts by looking at an individual project as a self-contained object, apart from its surroundings. Project aesthetics have been judged by considerations such as:

- Does the design visually express the project's functions?
- Are the details visually consistent?
- Do they support the total visual effect?

These and similar project characteristics are considered the internal aesthetics of a project. This is the first level of project aesthetics and is essential to a high-quality visual environment.

A second level of project aesthetics considers the visual relationships between a project and specific elements of its surroundings:

- Does the project contrast strongly?
- Does it block existing views?

These project characteristics are considered the relational aesthetics. They are the visual equivalent of good manners and can be very important to community acceptance of a project.

At the third and broadest level is environmental aesthetics. Here, the designer must examine the aesthetics of the total affected environment, of which any project is only a part. Do project visual characteristics:

- Enhance the quality of the environment?
- Decrease the quality of the environment?
- Affect the quality of the environment at all?

The designer should emphasise how to assess visual relationships between roadway projects and their surroundings and how to evaluate project effects on the quality of the visual experience in the project environment, as well as the internal aesthetics of projects.

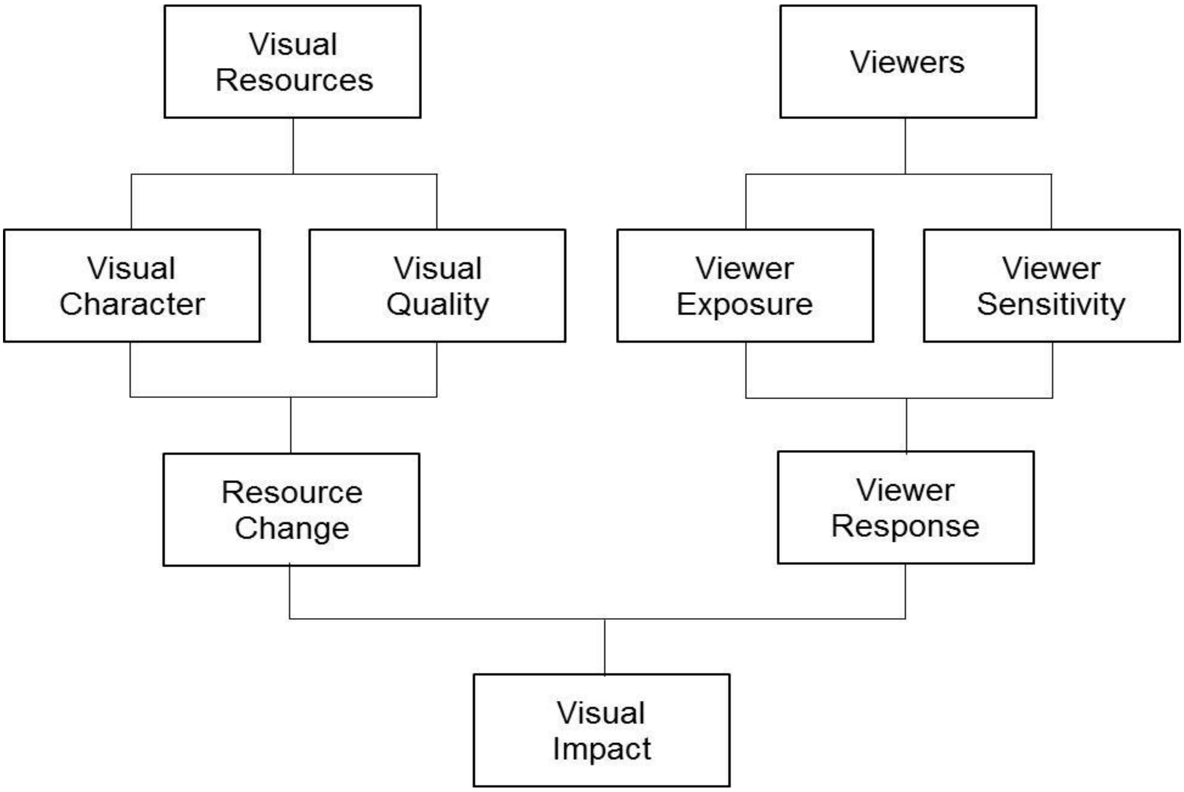
16.2.4.4 Visual Assessment Process

A generalised visual impact assessment process is illustrated in Figure 16-1. The major components of this process include establishing the visual environment of the project area, and identifying viewer response to those resources. These components define the existing or baseline conditions. The designer can then assess the resource change that would be introduced by the project and the associated viewer response; these allow the designer to determine the degree of visual impact.

16.2.4.5 Roadway Decisions with Aesthetic Implications

Figure 16-2 presents a checklist for the designer to use to evaluate how the roadway design may impact aesthetics. Specifically for landscaping, see Section 16.3.4.5.

Figure 16-1: The Visual Environment



16.2.5 Documenting and Reviewing Visual Impacts

The Abu Dhabi *Environmental Assessment for Road Projects Manual* (20) presents the policies for the content and preparation of the project environmental document. One of the topics that will be addressed in the document is the visual amenity and aesthetics impacts of the project. Visual effects are included within the document under the heading of “Visual and Aesthetics.” The following discussion provides guidance on the documentation of visual amenity and aesthetics.

A visual impact assessment for a large roadway project may be a considerable undertaking. Although this discussion may be a necessary element of the environmental studies for a roadway project, it may be too detailed for the environmental document, which should cover only those environmental issues that have a significant bearing on project decisions.

The narrative text in the environmental document should briefly describe:

- the principal visual characteristics of the project,
- the visual resources and viewers affected,
- the significance of the main visual issues,
- the effects of the project alternatives, and
- any mitigation measures.

Much of the discussion on visual impacts should be via graphic display. Graphic exhibits that are particularly helpful include the project view shed, photographs of key views, and illustrations of the project’s effect on these views.

Figure 16-2: Roadway Decisions with Aesthetic Implications

System Planning
- Design speed
- Capacity
- Access Control

Corridor/Location
- Alignment
  - horizontal
  - vertical
- Frontage roads
- Zoning
- Utility crossings
- Interchange location
- Intersections
- Joint development
- Urban vs. rural

Design
- Standards
- ROW width
- Sidewalks
- Pedestrian crossings
- Cycle paths
- Erosion control
- Clearing limits
- Median width
- Signing
- Pavement surface
- Slope treatment
- Culverts
- Ditches
- Sound barriers
- Rest areas
- Stream relocation
- Structures
  - bridges
  - walls
- Shoulder treatment
- Sight distance
- Roadside barriers
- Median barriers
- Landscaping
- Fencing
- Grading
- Lighting
- Advertising sign control

Construction
- Temporary erosion control
- Clearing practices
- Borrow pit operation
- Clean up
- Waste areas

Maintenance
- Mowing practices
- Litter pickup
- Painting
- Pavement maintenance

Operations
- Signing
- Pavement markings
- Lighting
- Traffic markings/lights
- Impact attenuators
- Delineators

The visual impact discussion should contain sufficient information on the visual characteristics of the project, those who will view the project, and the visual resources of the project area to support the findings of significance and effect. Evaluations should be supported by factual descriptions and illustrations. For example, an assertion that existing visual resources in the project area are low in visual quality should be preceded by a short description of these resources and representative photographs. Proposed mitigation measures should be logically related to adverse visual impacts or offsetting beneficial effects.

16.3 Landscaping

16.3.1 Landscape Design Principles

16.3.1.1 General

The Abu Dhabi *Road Landscaping Manual* (52) and the Urban Planning Council *Public Realm Design Manual* (13) present policies, procedures, and practices for roadway landscaping. The intent of the *Road Landscaping Manual* is to provide a clear understanding of the procedures and requirements associated with the planning, design, construction, and maintenance of the transportation system as it relates to the landscape. In addition, the *Manual* presents the role and responsibilities of the landscape architect and the importance of their relationship with the other roadway disciplines. Achieving these goals requires consideration to the natural, ecological, aesthetic, economic, and cultural influences related to the landscape.

The key components of the *Manual* are:

- Clear approach to the assessment, planning, and the design of the landscape.
- Need for the integration of the landscape within the roadway design.
- Need for landscape management and maintenance.
- Focus on sustainability.
- Provide standards to unify and regulate the landscape design process.
- Consider climate, ecological, and other environmental factors within the landscape design.
- Provide suitable plant material for Abu Dhabi's climatic regions.

Section 16.3 discusses landscaping within the context of the roadway design.

16.3.1.2 Objectives

Roadway aesthetics is an important consideration in Abu Dhabi's roadway development program. Plantation along a roadway positively affects the visual quality and it breaks the driving monotony and, hence, contributes to safety by providing visual interest to reduce driver boredom and fatigue. Figure 16-3, Figure 16-4, Figure 16-5, and Figure 16-6 show landscaping design along freeways, interchanges, urban streets, and roundabouts, respectively. In addition, the design and appearance of landscaping add value to positive urban design outcome. Furthermore, roadside vegetation has the potential to intercept pollutants from vehicle emissions and absorb contaminants; subsequently, improving local soil and air quality as well as surface and ground water management. For these reasons, landscaping within roadway corridors deserves careful management. The Emirate promotes best practices in landscape management within roadway corridors and integrates with water conservation policies to ensure that environmental stewardship keeps pace with water's increasing value and sustainable development of indigenous herbs, shrubs, bushes, and trees. Therefore, the following commitments are important:

- Ensure that landscaping maintains and improves but does not compromise roadway safety and operations, particularly clear sightlines to signs and delineation around curves.

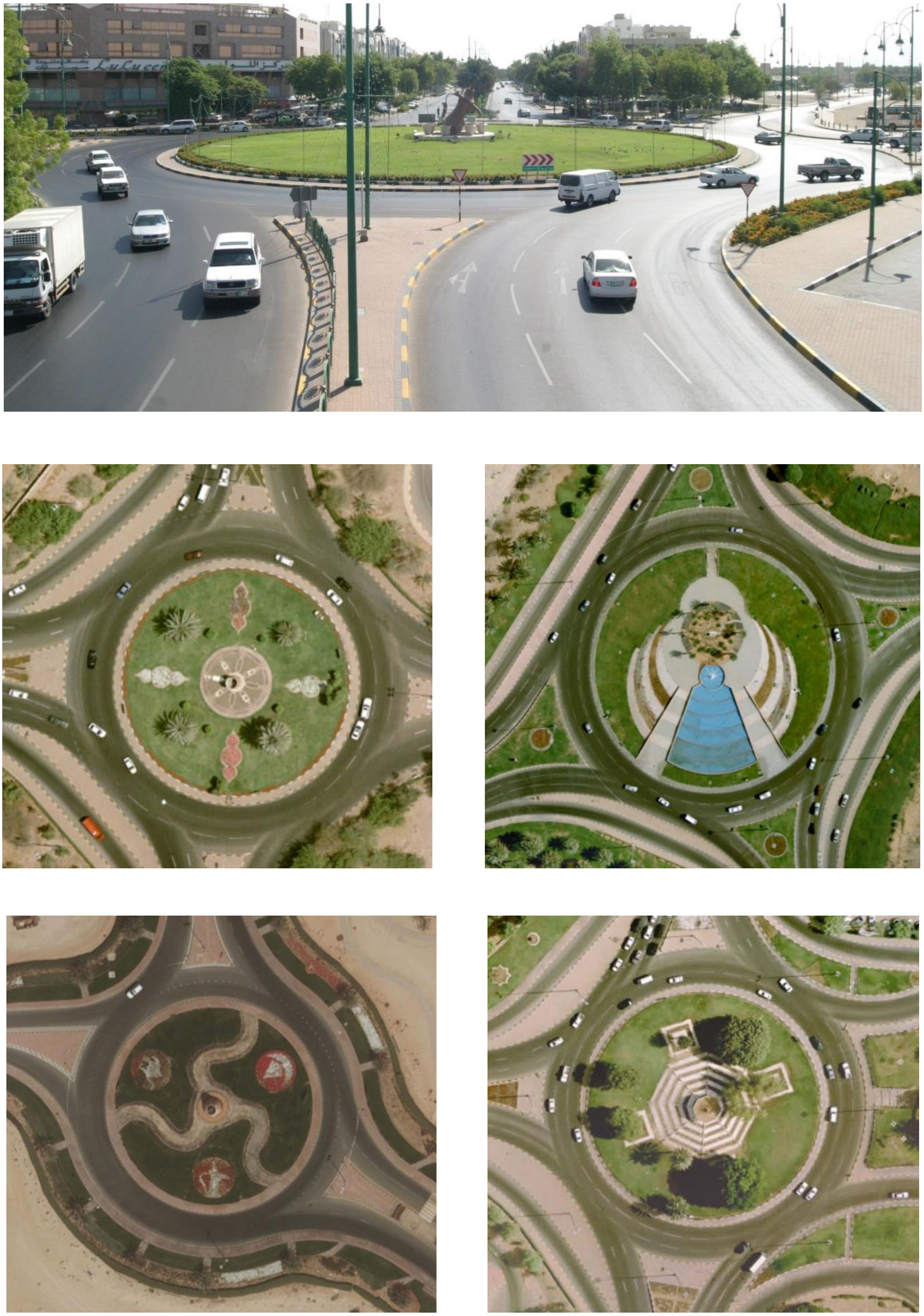
- Improve roadway aesthetics by landscaping. Particular attention should be paid to screening unpleasant/objectionable views.

Figure 16-3: Freeway Landscaping

Figure 16-4: Interchange Landscaping

Figure 16-5: Urban Street Landscaping

Figure 16-6: Roundabout Landscaping



- Ensure that planting schemes will not contribute to the strobing of roadways.
- Ensure appropriate indigenous and survivable plant selection within the roadway corridor to positively affect ecosystem integrity, to protect ecological values in adjoining land, and to help integrate the roadway into the surrounding landscape.
- Ensure landscaping has no negative effects on the pavement.
- Ensure the maintenance of the open area natural landscape. Vegetation shall be indigenous (not exotic) to harmonise with the local environment.
- Encourage creative landscaping designs, including use of appropriate hardscape where feasible.
- Ensure that the centre median shall have location-specific plants that shall not obstruct the motorists' view. The broad thick-leave, low-height bushes/shrubs/herbs shall be planted in the centre median to avoid opposite traffic glare at night and increase road visibility.
- Ensure that the watering mechanism shall make use of best available water conservation technology and shall not hamper the traffic flow or create a hazard through water spill.
- Ensure that hard trunk trees shall not be planted within a runoff/recovery zone and in the centre median to prevent fixed object crashes. Instead, establish frangible plant species/crash absorber plantation in those areas to make the roadside more forgiving.
- Provide vegetation that may be used to control soil erosion at selected locations.
- Ensure that landscaping projects shall have provisions for plant establishment periods for survival.
- Ensure that 80% of the total proposed planted areas along the roadside in any project must specify low water use plants.
- Ensure that up to 20% of the total project planted areas may specify moderate water use plantings. The moderate water use areas plantings should be installed near the highest pedestrian activity areas (e.g. intersection corners, mid-block crossings, transit stops) and in special focal areas as water management plan allows.
- Prevent no high water-use trees or plants of any kind in roadway rights-of-way unless special permission is obtained.
- Acknowledge in urban areas where right-of-way is limited, landscape must compete with other enhancement activities such as cyclists and pedestrian facilities.
- Use the Abu Dhabi *Road Landscaping Manual* (52) as the landscaping standard for all roadway landscaping design and maintenance.
- Consideration must be given to how maintenance workers and vehicles will access the landscaping and where will vehicles stop to unload mowers or pick up waste plant material like palm fronds etc.

16.3.1.3 Principles

The following controlling principles are based upon the conservation of natural resources; creating a facility that is compatible with its surroundings; minimising future management efforts and expenditures; and producing a high quality, environmentally responsible finished product:

1. Environmental Impact. Where practical, avoid adverse or disruptive impacts to landscape and environmental features on or adjacent to the right-of-way. Where total avoidance of adverse or disruptive impacts is not practical, undertake all reasonable measures to reduce and minimise impacts to these features. If damage or disruption is unavoidable, undertake all reasonable measures to offset damages through mitigation in the project area or other designated areas. Note that the designer cannot recreate or restore natural systems but can utilise native plant materials to represent some of the appearances and functions of the impacted feature.

2. Environmentally Sensitive Areas. Consider environmentally sensitive areas and those harbouring endangered species to be a controlling factor in all designs.

3. Use of Indigenous Plants. Emphasise the use of plants native to and grown in Abu Dhabi, which are appropriate to the site, its planned use, and its future management.

4. Site Compatibility. Design a specific landscape effect that is compatible with the site.

5. Future Management Considerations. Consider the future plan for the roadside area to be a controlling factor in the planning and design of that area.

6. Sustainable Roadside Environment. Strive to produce a sustainable roadside environment.

7. Visual Quality. Visual appearance and visual unity of the facilities are important components. Recognise that visual quality must be a component in almost all design development and that numerous factors influence the final appearance of the finished project. Durability and appearance are the two items most noticed by the travelling public.

16.3.1.4 Application of Principles

Apply landscape and environmentally based design principles to the full range of roadway types. They also apply to all roadway components and features including:

- the roadway (i.e. the travel surface itself);
- the roadside (i.e. the remainder of the right of way including any existing natural vegetation and/or plantings);
- ancillary structures (e.g. bridges, culverts, retaining walls); and
- roadway appurtenances (e.g. fences, signs, lights, traffic barriers).

The extent of the application of these principles will depend on the type of project, the environmental resources affected, and the public entities involved.

16.3.2 Overview of Landscape Design Process

16.3.2.1 Planning and Location Phase

During the planning and location phase, apply the landscape and environmentally based design principles to assess environmental effects and identify measures to mitigate adverse impacts. Use the principles to help identify landscape features that can be incorporated into project planning and to influence development of alternatives to the proposed action and any environmental commitment.

16.3.2.2 Design Phase

In the design phase, focus on how each of the principles apply to a particular project, what commitments have been made, and how to incorporate the principles into design plans for such project features as landscaping, environmental impact mitigation measures, support facilities, and associated structures.

16.3.2.3 Project Construction

The construction phase ensures that mitigation measures committed to in the design and planning phases are implemented and conditions discovered in the field receive consideration.

16.3.2.4 Operation and Maintenance

Finally, the incorporation of landscape and environmentally based design principles into the operation and maintenance of transportation facilities can help to ensure the continued effectiveness of project mitigation measures.

16.3.3 Responsibility for Preparation of Design

16.3.3.1 Roadway Designer

It is the designer's responsibility to be aware of any aspect of the design that would adversely affect the roadside and to modify such design elements, where necessary, to achieve a harmonious design that meets the criteria for the project. The designer also is responsible for notifying the DMT Landscape Architect of potential impacts on their operations.

16.3.3.2 Abu Dhabi DMT Landscape Architects

The DMT Landscape Architect provides the required expertise in landscaping aesthetics, material selection, and visual quality. The Landscape Architect advises the designer and reviews design criteria and proposals based on the Abu Dhabi *Road Landscaping Manual* (52).

16.3.3.3 Abu Dhabi Environmental Agency

The Abu Dhabi Environmental Agency identifies those aspects requiring the input of the Landscape Architect, including commitments regarding wetland mitigation, erosion control, endangered species habitat protection and restoration, and tree replacement or removal. This Agency also identifies the impact of the visual component of noise mitigation structures and notes critical areas adjacent to the project needing protection.

16.3.4 Design Factors

16.3.4.1 General

Design factors are intended as a guide for the designer to use in developing a solution for a particular project condition. Determine the appropriateness of the design as it applies to the given landscaping features of the roadway environment.

16.3.4.2 Coordination of the Design Process

Use the following guidelines to coordinate the design process with respect to landscaping:

1. Internal Coordination. During the design process, coordination of many disciplines (e.g. engineering, landscape architecture, hydrology) is needed to achieve the proper environmentally based design. This is true not only for large and complex projects but also for small and simple projects. Obtain all available inputs to ensure a coordinated environmentally based design.

2. Coordination with Local Jurisdictions and the General Public. The designer is responsible for coordinating with local jurisdictions (e.g. DMT, DMT, AAM, WRM) and abutting landowners adjacent to or affected by the project. Consider the potential impacts to any local or long range plans. Ensure that aspects of the project are not adverse to broad public values. Early coordination with local jurisdictions and with the public may provide valuable input to ensure the success of an acceptable design. Coordination at the local level includes municipalities, residential and commercial developments, and emergency services.

16.3.4.3 Protection of Existing Features

Certain existing landscape features, whether manmade or natural, should be protected through a process of identification; enhancement, restoration, or preservation; and avoidance or incorporation into the design of the roadway improvement. Consider the following when determining the need for protecting existing features:

1. Review Previous Commitments. Review commitments in environmental documents, tree surveys, project reports, and other project documents for those landscape features requiring protection during project development and implementation.

2. Statute Protection. Determine which features of the project area are protected by statute. These may include endangered species, natural areas, and cultural sites.

3. Cultural Environment. Determine the project's setting or cultural environment (e.g. rural, urban, a transitional area). The design should be influenced by the cultural and physical environment adjacent to it and existing features should be protected where practical.

16.3.4.4 Grading and Alignment

That portion of the design process concerning alignment and grading offers the best opportunity to fit the roadway into the landscape, thereby avoiding unnecessary environmental impacts and yielding a functional and aesthetically pleasing form. The AASHTO *Guide for Transportation Landscape and Environmental Design* (59) provides guidance. In addition, consider the following guidelines:

1. Environmental Commitments. During preliminary design, ensure that all environmental commitments are reviewed to appropriately influence alignment and grading decisions. These commitments also will serve as controls during detailed plan preparation.

2. Surrounding Landscape. Give consideration to the surrounding landscape. Blend the alignment and grading to fit the existing topography with minimal visual or physical disruption.

3. Clearing and Construction Limits. Carefully plan the establishment of clearing and construction limits. Consider both existing landscape features and critical areas.

4. Plant Survival. Consider the survival potential of existing plantings to be preserved and proposed plantings at the time that grading decisions are made. This is especially critical in confined areas where landscape features are proposed. Plant material to remain within the project limits should be properly cared for so that it is alive and in good condition when the project is complete. Consider the following guidelines:

    a. Root Pruning. Specify root pruning where trenching or excavation is within the root zone of adjacent trees or shrubs to remain in place to prevent ripping up roots.

    b. Fertilizer Nutrients. Specify fertilizer nutrients for trees and shrubs that will be disturbed by construction but will remain-in-place.

    c. Supplemental Watering. Specify supplemental watering for trees and shrubs that will be disturbed by construction but will remain-in-place. Watering should begin immediately after root pruning, top pruning, or other construction disturbance.

    d. Tree Pruning. Specify tree pruning where an entire tree needs to be pruned, to correct structural problems, or improve the overall appearance.

    e. Temporary Fencing. Specify temporary fencing where protection of existing trees is necessary.

    f. Tree Trunk Protection. Specify tree trunk protection to prevent damage by construction equipment to existing trees located within or along the construction area.

16.3.4.5 Visual Quality

A project's visual quality is ensured by encouraging a positive visual change that will improve or enhance the surrounding landscape. Define the visual environment by identifying key views, analysing resources, depicting the project's proposed appearance, and assessing its visual impacts. To better provide for visual quality in a project, evaluate the project's relationship with the following:

- natural landscape elements,
- topographical and physical characteristics,
- ecological influences,
- recreational sites,
- residential areas and their character,
- historical features,
- visual values,
- existing land uses (e.g. industrial), and
- existing and proposed project profile.

Review these elements to ensure that visual quality is adequately integrated into the project. See Section 16.2 for additional information incorporating visual quality.

16.3.4.6 Safety

Safety should be the highest functional goal of every design, and all landscape and environmentally based design principles must be compatible with this criteria. During design, consider the following with respect to landscaping:

- the location, size, and height of plantings in relation to sight distance, drainage, and roadside clear zones;
- traffic-calming designs in urban areas; and
- pedestrian safety in areas such as rest areas, transit stations, and cycle paths.

16.4 Roadway Lighting

Lighting may reduce night time crashes on a highway or street and improve the ease and comfort of operation thereon. Statistics indicate that night time crash rates are higher than daytime crash rates. This may be attributed to reduced visibility at night. There is evidence that in urban areas where there are concentrations of pedestrians and roadside intersection interferences, fixed-source lighting tends to reduce crashes. Lighting of rural highways may be desirable, but the need for it is much less than on streets and highways in urban areas. The general consensus is that lighting of rural highways is seldom justified except in certain critical areas, such as interchanges, intersections, railroad grade crossings, long or narrow bridges, tunnels, sharp curves, and areas where roadside interference is present. Most modern rural highways should be designed with an open cross section and horizontal and vertical alignment of a high type. Accordingly, they offer an opportunity for near maximum use of vehicle headlights, resulting in reduced justification for fixed highway lighting. See the Abu Dhabi *Road Lighting Manual* (46) for details.

16.4.1 Utility Poles

Utility supports (poles) should be placed outside the roadside clear zone whenever possible. See the Abu Dhabi *Roadside Design Manual* (28) for the appropriate clear zone dimensions for the various functional classifications. Because of lower speeds and parked vehicles, there is much less chance of injuries to vehicle occupants from striking fixed poles on a street as compared to a highway. Poles should not be erected along the outside of curves on ramps where they are more susceptible to being struck. Poles located behind longitudinal barriers (installed for other purposes) should be offset sufficiently to allow for deflection of the longitudinal barriers under impact.

16.4.2 Facilities

Providing lighting for all highway facilities is neither practical nor cost effective. It is generally Abu Dhabi DMT practice to only provide highway-facility lighting where justified based on sound engineering judgment and on the criteria, guidelines, and principles presented in the Abu Dhabi *Road Lighting Manual* (46).

A location that appears to justify lighting does not necessarily obligate the Department to provide funding for a lighting project. The Department will determine the economic feasibility and identify candidate locations for lighting projects. Agencies with jurisdiction over other facilities may determine the feasibility of providing lighting for their facility.

For a highway facility to be considered for lighting, the lighting system must be both economically feasible and justified based on the applicable criteria. The impacts of local conditions (e.g. roadway geometry, ambient lighting, sight distance, signing) also should be considered when analysing highway lighting needs.

16.5 Fencing

16.5.1 General

Fencing is desirable along high-speed roadways to protect the driver from unexpected intrusions from outside of the right-of-way line. Fencing deters unauthorised and unsafe entry to the roadway by vehicles, pedestrians, or animals. It also reduces the occurrence of objects being dropped or thrown from roadway overpasses.

Unless warranted for roadway applications, fencing is normally the responsibility of the abutting property owner. Fences may be necessary for retaining livestock, discouraging trespassing, defining property boundaries, or otherwise to keep land use activities within bounds. If private fences are impacted by a roadway project, their relocation or disposition is reconciled as part of the right-of-way negotiations and settlement.

16.5.1.1 Location

Fencing is typically provided along all segments of fully access controlled facilities; along certain segments of expressways; near schools, playgrounds, and parks; near livestock areas; on some bridges; and between frontage roads and access controlled roadways. Fencing is usually erected parallel to the roadway centreline. Where right-of-way lines are irregular, the fencing should still be basically parallel to the roadway, if the fencing is within the roadway right-of-way. In general, the fence line should coincide with the right-of-way line; however, deviations are acceptable if justified. See the Abu Dhabi *Standard Drawings for Road Projects* (27) for construction details.

16.5.1.2 Fence Types

The following fence types are used in Abu Dhabi:

1. <u>Woven Wire</u>. This fence type consists of a woven wire mesh and two strands of barbed wire at the top for a total height of 1.2 m.

2. <u>Chain Link</u>. Chain link fence consists of an interlocking wire mesh on metal posts. It is typically 1.2 m or 1.8 m high.

3. <u>Camel Fence</u>. Where a fence is required for camel crossings in sand areas, a modified strained wire fence has been developed that incorporates two 19-mm diameter cables as a deterrent to camel crossings or cutting. This fence is more open than other types of fence that have a tendency to trap paper and debris, which can cause sand depositions. In areas of active dune movement, provide a stabilized strip of at least 1 m along fence lines. See Section 3.7 for additional guidance on sand control.

Where a portion of an existing fence, which differs from the standard type or height, is to be replaced, the new fence should match the portion of the fence that will remain in place.

Section 16.5.2.4 discusses the criteria for installing gates along access-controlled facilities. Gates may also be provided where fencing must cross the road to enclose livestock or at entrances of farms. Coordinate with the landowner to determine the type of gate that should be used.

16.5.2 Freeways and Expressways

The following will apply to fencing along freeways and expressways.

16.5.2.1 Warrants

Fencing is required along all fully access-controlled facilities. Provide continuous fencing along either the right-of-way or access control lines. Also, provide fencing for the entire limits of access control along the crossroad and along the first access connection. However, engineering judgment may dictate exceptions. In addition, where retaining walls, concrete barriers, sound barriers, sight screens, etc., are used, fencing usually is not required to preserve access control.

For partial access-controlled facilities (i.e. expressways), fencing is highly desirable. At a minimum, existing fencing should be replaced in rural areas and should be considered in built-up areas in rural locations. Also, to minimise future access problems along crossroads, provide fencing along the entire access control limits of each crossroad.

16.5.2.2 Location

Fences generally will be located to coincide with the right-of-way line, except for the following:

1. Frontage (Service) Roads. Where service roads exist or will be constructed, place the fencing between the adjacent roadway and the roadway or ramps. Fencing is normally located outside the clear zone of a roadway. Where access control fencing is located between a service road and the freeway, it is not necessary to provide fencing outside the right-of-way line of the service road.

2. Structures. Wherever railroads or grade separations are encountered that prevent the fence from being continued on a straight line, tie the fence to the structure, abutment, or wing wall to ensure that full access control is maintained.

16.5.2.3 Construction

Construct access control fencing on the right-of-way line with the face of the fencing toward the abutting property. Delineate this location on the contract plans.

16.5.2.4 Gates

Where considered necessary for maintenance and operational purposes, gates may be required in the access-control fencing. Limit the number of these gates to an absolute minimum. Only provide gates to allow access for:

- Maintenance to otherwise inaccessible portions of the roadway right-of-way, and
- Public utility installations where no other access is available or can be reasonably provided.

All gates must have adequate locking devices to ensure that they are only used by authorised personnel. The responsible government agency must maintain a record of all personnel that possess keys to the locked gates.

16.5.3 Pedestrian Fencing

Pedestrian fencing may be used for the following purposes:

- to prevent pedestrians from crossing road at high risk locations (e.g. mid-block crossing on major streets with medians);
- to assist in directing pedestrians to safe road crossings (e.g. at roundabouts);
- to prevent pedestrians from walking into the path of vehicular traffic, particularly at locations where pedestrian activity is high; and
- to exclude pedestrians from high-volume, high-speed roads (e.g. freeways, expressways).

There is wide range of fencing that may be used to constrain pedestrians. The designer should consider the following:

- Fencing should be constructed principally from vertical members to make it difficult to climb.
- Avoid ridged horizontal members to limit the possibility of components spearing through a vehicle in the event of a collision.
- Fencing located in the clear zone must not be hazardous to either vehicle occupants or pedestrians when struck by a vehicle.
- The area between the roadway and pedestrian fence should be designed using appropriate landscaping or surfacing to discourage its use by pedestrians.
- Pedestrian fencing used to constrain pedestrians should be design to not resist penetration by a vehicle. If vehicle constraint is required, use an appropriate safety barrier; see the Abu Dhabi *Roadside Design Guide* (28).

16.5.4 Protective Fencing on Roadway Overpasses

16.5.4.1 General

Protective fencing is generally provided:

- where there is a potential hazard to roadway users resulting from objects being dropped, thrown, or dislodged from overhead crossings;
- where there is a need to protect pedestrians who cross on the overpass (e.g. near schools, playgrounds, or other locations where children may be present);
- where a cycle path is carried across the structure;
- where problems with pedestrians cannot be easily kept under surveillance by police;
- where experience from nearby structures indicate a need;

- where there is a need to protect private property located underneath the structure (e.g. power stations, buildings, railroads).

The protective fencing should satisfy the aesthetic considerations of the structure and comply with the applicable structural requirements.

16.5.4.2 Pedestrian Bridges

Provide protective fencing on all new pedestrian bridges. The need for protective fencing on existing pedestrian bridges will be determined on case-by-case basis using the guidance discussed above.

Either a semi-circular enclosure or a partially enclosed curved top fence are the most effective in discouraging objects being thrown from the overpass.

16.5.5 Fencing or Safety Devices on Top of Retaining Walls

One purpose of a fence is to alert pedestrians, cyclists, and/or motorists of a difference in elevation to prevent injury from a fall or vehicular crash. Consider installing a fence at the top of any wall that is over 300 mm tall if the top of the wall is closer than 600 mm to a sidewalk, trail, parking lot, or stairway landing. Walls located further away from human or vehicular activity may be higher before a fence is considered necessary. Regardless of the height of the wall, provide a fence if any activity (e.g. driving, walking, riding a cycle, running) could result in harm or damage by someone inadvertently going over the wall. A fence is required when the difference in grade level on either side of the wall is in excess of 1.2 m. Consider the aesthetics of any fence, especially in urban areas, where the wall and barrier are located adjacent to private property.

16.6 Rest Areas

16.6.1 General

Rest areas are an essential element of the complete roadway system. They are provided for the safety and convenience of the travelling public. A rest area provides an oasis for fatigued travellers to relax and refresh from the rigors of roadway travel. A comfort station is provided to accommodate the basic needs of the travelling public. Multi-use land may also be provided for picnic tables and limited recreational activities.

16.6.2 Location of Rest Areas

16.6.2.1 Spacing

Select and acquire the site for the rest area concurrently with the acquisition of the freeway right-of-way. The average spacing between rest areas is approximately 80 km to 100 km. However, many factors influence the final location of the site.

16.6.2.2 Site Selection

Rest areas should be developed as part of a comprehensive program that considers:

- the attractiveness of the location,
- the site's topography,
- the distance from other rest areas,
- the distance between interchanges, and

- the availability of water and utilities.

Consider the following additional issues during the site selection process:

1. Needs Assessment. The designer will make the determination that a rest area is to be constructed within certain limits of a roadway corridor.
2. Site Investigation and Prioritisation. The client will investigate potential sites and prioritise the sites using the form provided in Figure 16-7. Site selection basically considers the scenic quality of the area, access to the area, and the site's adaptability to development including available utilities.
3. Residential and Industrial Areas. Do not locate rest areas near residential or industrial areas due to noise and fumes.
4. Water and Sewer Needs. As practical, use a local jurisdiction's service for the facility's water and sewer system. If access to a local jurisdiction's system is unavailable, provide well water and an on-site waste disposal system.
5. Right-of-Way Needs. Approximately 10 hectares of right-of-way are necessary to accommodate each site.
6. Locations Near Interchanges. Where a rest area is located near a freeway interchange, provide a minimum distance of 1000 m between the ramp gores of the rest area and the interchange.
7. Entrance and Exit Ramps. Design rest area entrance and exit ramp terminals according to the typical design criteria used for interchange ramps. See Section 16.6.3.

Figure 16-7: Evaluation Criteria for Site Selection, Layout, and Design of Freeway Rest Areas

Alternatives

Criteria	#1	#2	#3	#4	#5
A. Aesthetics					
1. Fits Topography					
2. View From Freeway					
3. View From Rest Area					
4. Landscaping Potential/Screening					
5. Natural Site Features/Existing Tree Cover					
B. Geometrics					
1. Adequacy Truck/R.V. Parking					
2. Adequacy Car Parking					
3. Expansion Potential					
C. Use Areas					
1. R.V. Picnic Area					
2. Picnic Areas					
3. Sidewalks					
4. Playground					
5. Pet Walks					
6. Security					
D. Land Use Compatibility					
1. Adjacent Land/Subdivisions					
E. Environmental Control					
1. Water Quality					
2. Endangered Species					
3. Air					
4. Noise					
5. Right-of-Way					
F. Utilities					
1. Sewage					
2. Water					
3. Gas					
4. Electric					
G. Cost					
Totals					

Rank 1 -5
1 - Lowest
5 - Highest
N/A - Indicates Not Applicable

16.6.3 Design of Rest Areas

16.6.3.1 Vehicle Capture Rates

The criteria for estimating the number and type of vehicles that will enter a rest area is based on the Average Daily Traffic (ADT) and the percentage of heavy vehicles in the traffic mix. The criteria for determining the number of persons projected to use the rest area, the anticipated demand for water, and the type of rest room facilities that should be provided are found in the AASHTO *A Guide for the Development of Rest Areas on Major Arterials and Freeways* (60).

16.6.3.2 Design Elements

Consider the following guidelines in the design of rest areas:

1. Buffer Zone. To enhance patron safety, locate the facility a minimum distance of 100 m away from the freeway to create a buffer zone to the nearest use area.
2. Parking. See Section 16.6.3.4 for information on vehicle parking requirements.
3. Internal Roadways. Design internal rest area roadways for a minimum speed of 30 km/h and a desirable speed of 40 km/h.
4. Curbing. Curbing may be used on internal roadways to restrict illegal parking on the shoulder and to minimise vehicular encroachment.
5. Fencing. Fence the rest area right-of-way to prevent access to/from adjacent properties.

16.6.3.3 Entrance and Exit Ramp Terminals

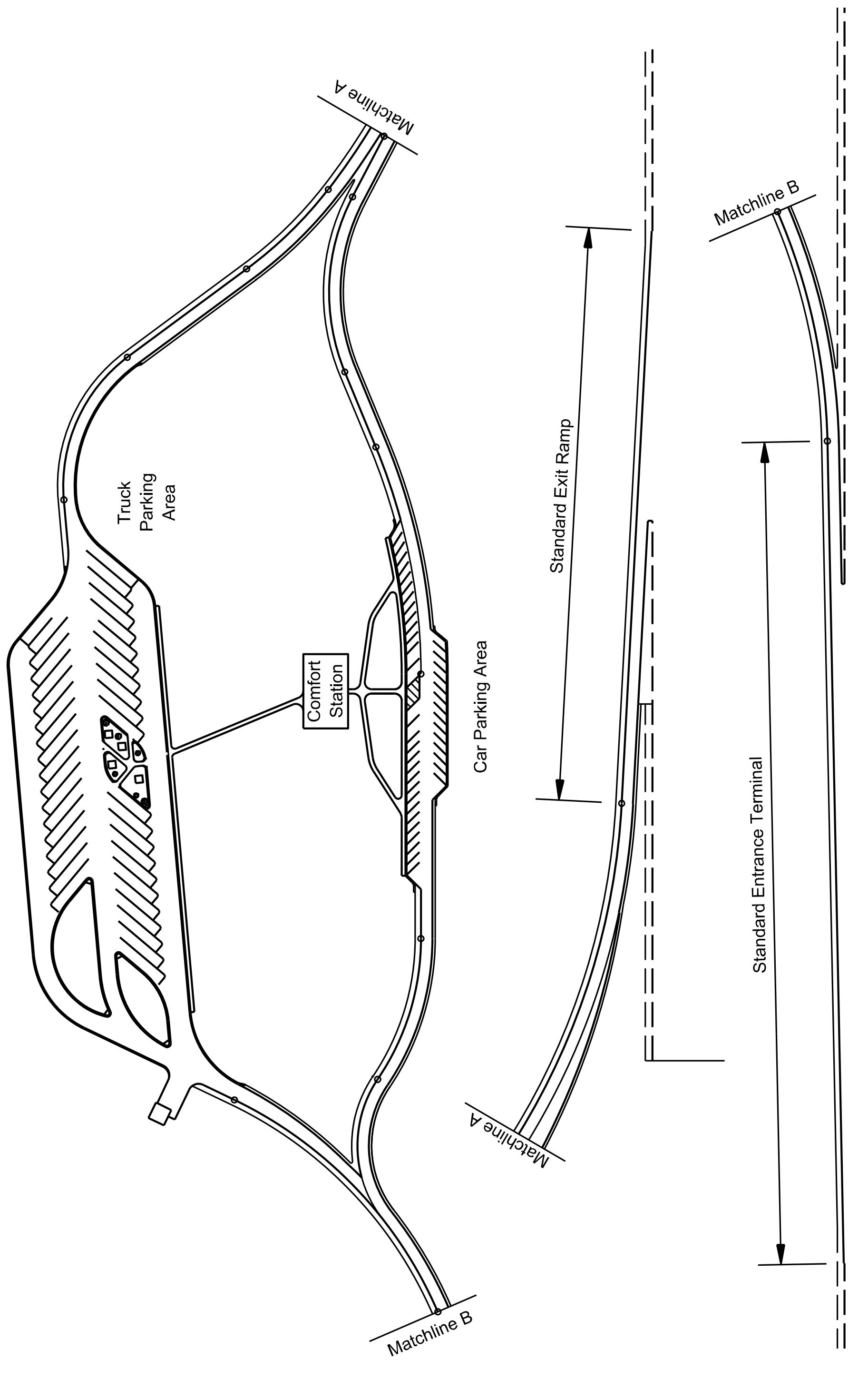
Provide access to/from rest areas according to the typical ramp terminal designs presented in Chapter 12 “Interchanges” and the Abu Dhabi *Standard Drawings for Road Projects* (27). Also, consider the following additional information on entrance and exit ramp terminals:

1. Exit Ramps. Design initial ramp curves for 80 km/h and limit superelevation to 5% maximum. Provide decision sight distance for the exit manoeuvre. Provide a deceleration distance of 180 m to 200 m from the gore of the exit ramp to the car/truck divergence gore.
2. Entrance Ramps. Provide a minimum distance of 350 m from the truck parking area to the gore nose on the entrance ramp. This distance typically will allow a truck to accelerate to an acceptable speed before entering the freeway.

16.6.3.4 Vehicle Parking Requirements

See Figure 16-8 and Figure 16-9 for typical design layouts. Typically, truck parking areas are located to the rear of a site. An exception to this would be if the terrain or a scenic vista would be better served if passenger cars were located at the rear of a site. Within the car parking area, the through roadway width of 6.0 m is desired with diagonal parking generally on both sides of the roadway. Provide curb and gutter within the vehicle parking area.

Figure 16-8: Typical Rest Area Design Layout



The truck parking area should typically provide a centre aisle 10 m wide with 45-degree diagonal parking on both sides; however, 35-degree may be used in confined areas. Design exit aisles to be 8.5 m in width, minimum, with an adjacent curbing. Design ramp widths and roadway geometry to accommodate a WB-20 design vehicle. Where WB-33D trucks are allowed, ensure the WB-33D design vehicle can be accommodated. Ramp widths are normally 5 m wide; however, increase this width to accommodate truck off tracking when providing a curved alignment. Figure 16-10 illustrates typical parking configurations for a WB-20 tractor-trailer truck.

Figure 16-9: Typical Rest Area Design Layout

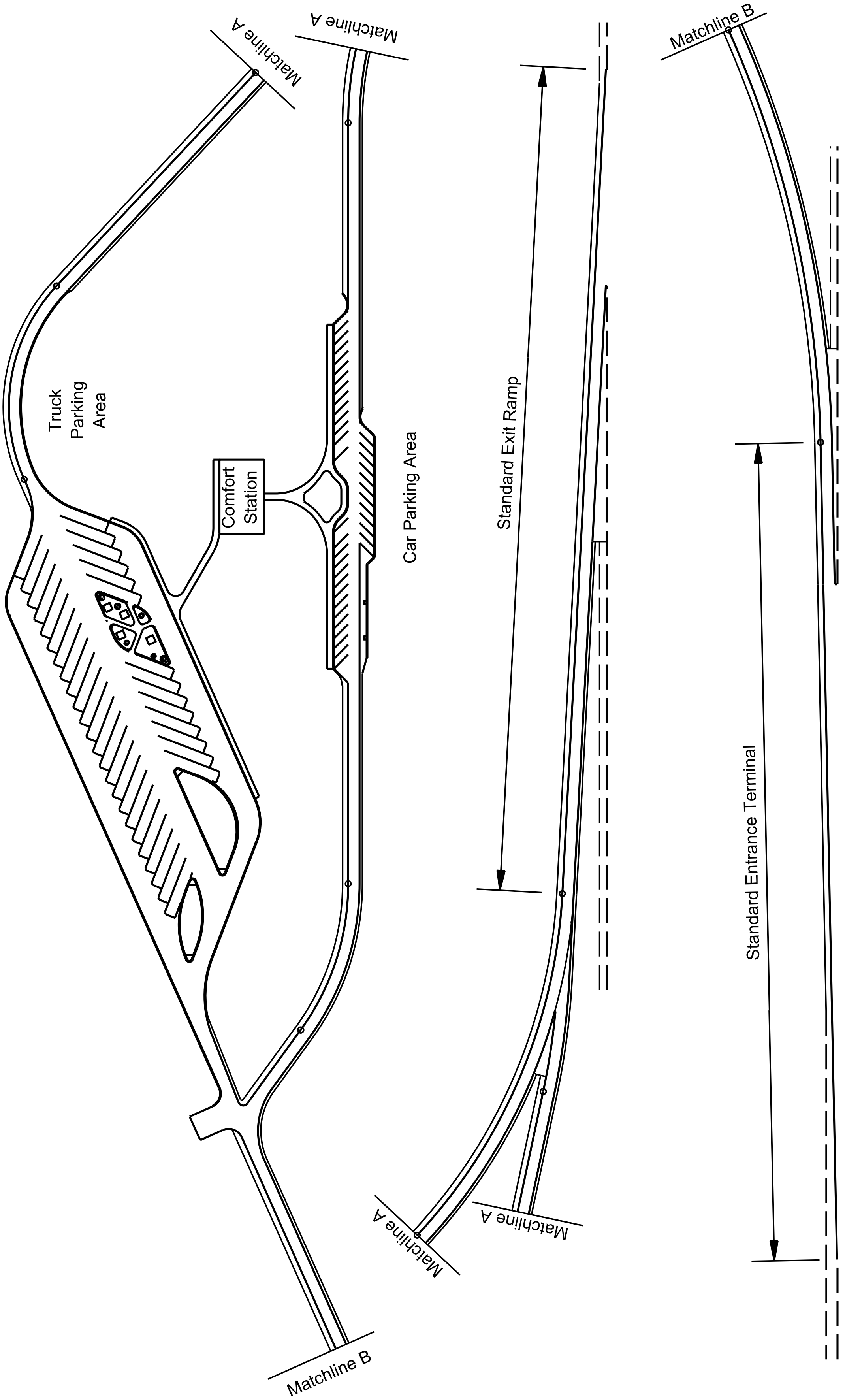
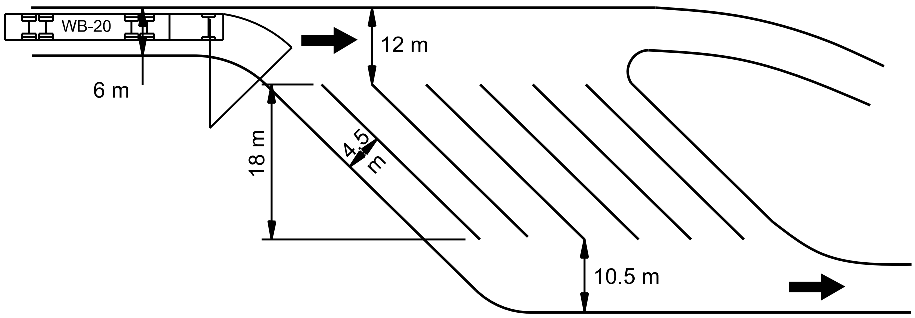
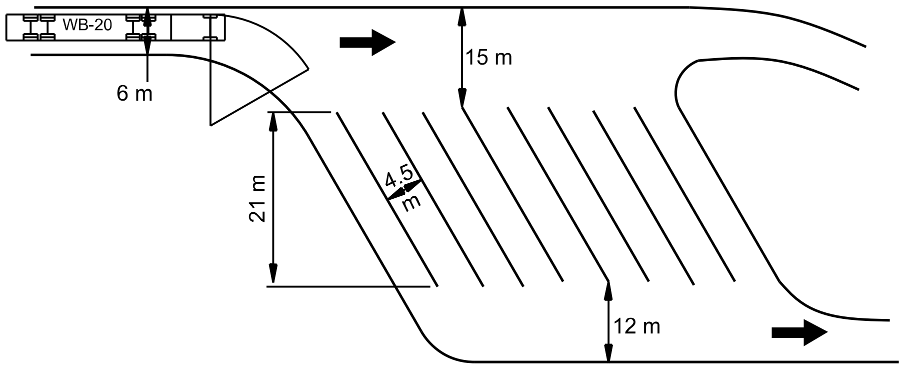


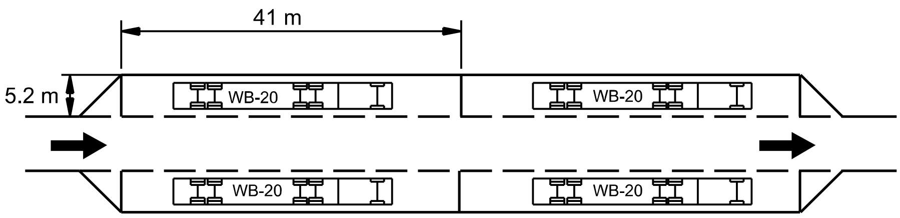
Figure 16-10: Typical Truck Parking Layout



45° Truck Parking



60° Truck Parking



Truck Aisle Parking

Note: All dimensions shown are minimums.

16.6.3.5 Pavement Design

Design the pavement section for the rest area exit and entrance ramps, roadways, shoulders, and parking areas according to the Abu Dhabi *Pavement Design Manual* (55). The concentration of heavy trucks braking on the ramps and inner roadways and the sharp turning manoeuvres to enter parking stalls requires these facilities to be considered “high stress” locations. Design the ramp pavement structure to handle overflow truck parking on shoulders.

Because of the likelihood of vehicle off tracking onto ramp shoulders, provide full-depth, paved, 2.4-m wide shoulders on the right, and fully paved 1.2 m wide shoulders on the left.

16.6.3.6 Building Facility

Entranceways to building facilities should provide an unobstructed walkway to the facility and a clear view into the lobby area. Elevated planters, walls, shrubs, and bushes should not obscure the view of the approaching patron. The open view also allows police to scan the lobby in a drive-by and record video surveillance. The following provides additional information on the design of building facilities:

1. <u>Vestibule</u>. The entryway should be a glass enclosure that provides an unobstructed view into the lobby. Use double air-lock doors with high-quality latches and closure hardware to withstand high use.

2. <u>Lobby</u>. The lobby area should provide approximately 80 m^2 to 100 m^2 of floor space with a generous amount of windows to maximise natural light. Windows should be provided with safety glass and sectioned in 1.2-m increments or less for ease of repair and maintenance. Protect full-length windows on the inside by placement of handrails and/or bench seating. Locate water fountains away from the windows to prevent splashing. Recessed lighting with vandal-proof flush lenses is desirable. Design emergency lighting to be flush with the ceiling.

3. <u>Vending Area</u>. Locate the vending area to the side of the lobby. Bench seating is desirable in this area. A security camera should be installed to monitor the vending area.

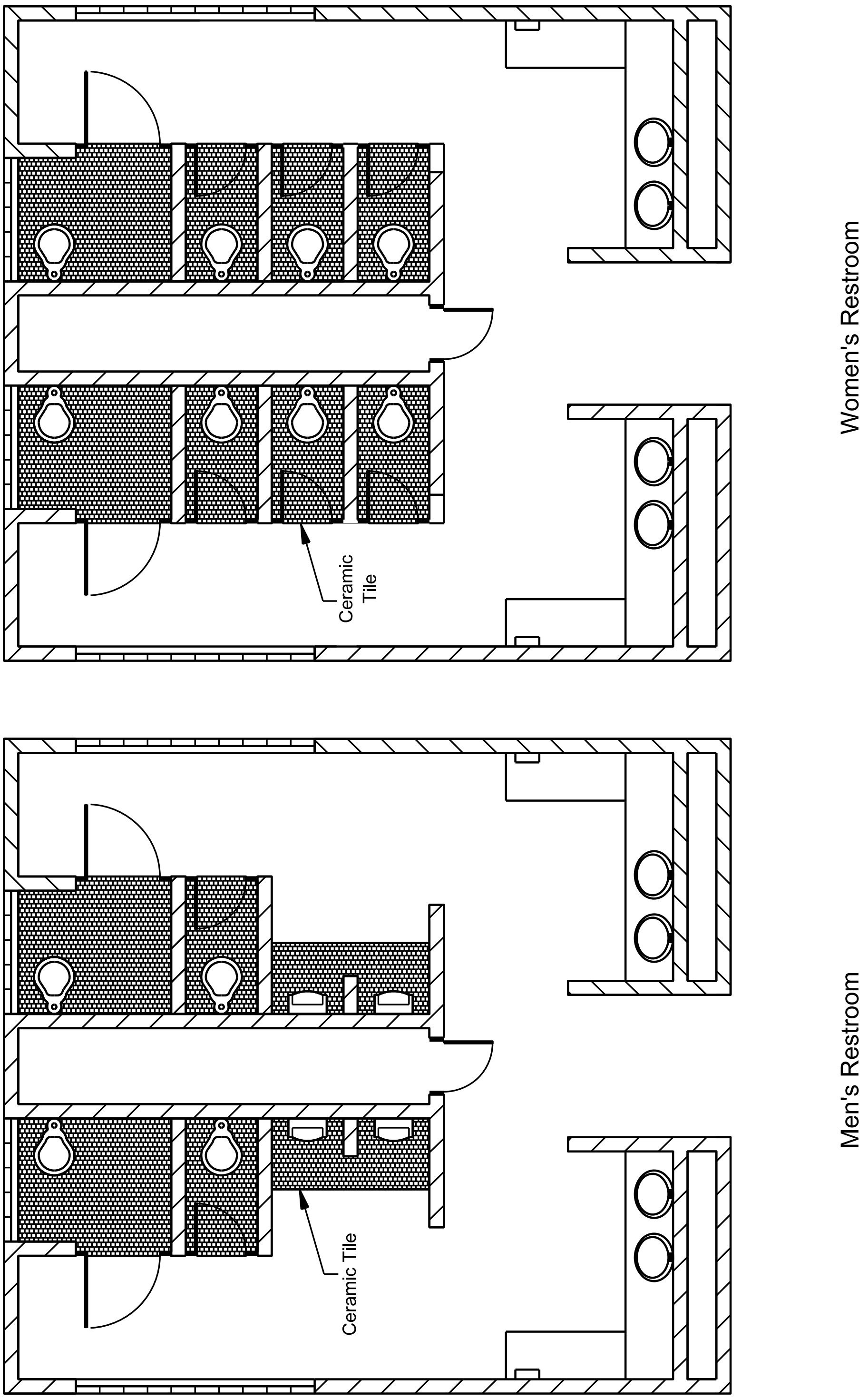
4. <u>Wall Space for Information Boards</u>. All rest area facilities should display a large map in a prominent location on either a wall or a centre kiosk. Additional wall space of approximately 3 m^2 is necessary for information services and notices.

5. <u>Telephones</u>. Provide a minimum of one telephone in the building facility lobby for public use. Additionally, one phone is required in the mechanical room. One phone is required for the attendants. Provide remote ringers in the lobby and outdoors for the mechanical room phone.

6. <u>Vending Storage Room</u>. The vending storage room should be located near the vending area with internal and external access doors. The room should be approximately 20 m^2 to 25 m^2.

7. Rest Rooms. Rest rooms should be a split design with open doorways (see Figure 16-11). Rest rooms with eight fixtures should be approximately 65 m^2. Design the rest room entry to prohibit direct sight lines into rest rooms from the lobby. The split design allows one-half of the rest rooms to be closed for cleaning and repairs. Plumbing and electrical systems should be valved and switched, respectively, to allow repair work in one-half of each rest room. The following provides additional information on rest rooms:

Figure 16-11: Typical Rest Room Layout



- Standardisation of components (e.g. faucets, valves, fixtures, washers, lighting, locks) are essential for ease of replacement. High-quality, heavy-duty vandal proof components are required.
- Rest room walls should be tiled with a neutral colour for ease of maintenance.
- Construct toilet stall partitions with minimum 100-mm, glazed-tile concrete blocks.
- Recessed toilet paper holders are desirable. Provide three-quarter length stainless steel stall doors with heavy-duty hinges and latches.
- Toilet stools will be of high quality, institutional type, floor-mounted, rear-discharge fixtures.
- Specify use of water-saving, automatic-flushing devices with an override.
- Provide an infrared sensory device at accessible washbasins.
- Include a small plate glass mirror over washbasins.
- Specify and locate heavy-duty hand dryers near the lavatories to minimise a slippery floor condition.
- Provide heavy-duty soap dispensers that will not drip onto floor.

8. Lighting. The lighting of a rest area may be provided by a combination of high-mast lighting and pole lights. See the Abu Dhabi *Road Lighting Manual* (46) for details. Pole lights that are 15 m high with high-pressure sodium luminaires are most appropriate to illuminate entrance and exit terminals, as well as ramps leading to and from the parking areas. The high-mast lighting is normally applicable to parking and multi-use areas with poles of 30 m in height. Where practical, provide lighting with fixtures mounted above ground in inconspicuous locations.

9. Sidewalks. Place sidewalks adjacent to all curb parking areas. Entrance walks to the building facility should provide direct access from the parking area and should not be obstructed by signing, bench seating, or planting areas. Accessories (e.g. drinking fountains, signing, newspaper containers, recycling receptacles) should be positioned adjacent to the sidewalk and anchored to concrete pads.

   Sidewalks should be 1.5 m wide and constructed of concrete with a slip-resistant surface. Other recreational walkways may be constructed with loose gravel, shredded bark, or similar loose material, to allow extended walks to other natural areas of the facility. Provide nature trails where site conditions warrant such amenities.

10. Mechanical/Storage Room. This room will house all of the mechanical equipment necessary to serve the facility. Approximately 30 m^2 to 35 m^2 is required with access doors to both the interior lobby and the exterior. Also, include a desk, telephone, and an area for security system computer and monitors in the room. Storage space is required with shelving for toiletry items, cleaning liquids, appliances, and electronic equipment for the security system.

16.6.3.7 Other Amenities

The multi-use area provides a restful environment for the traveller and allows some limited recreational activities. Many facilities offer a scenic vista to the adjacent landscape. Use the following guidelines to provide the most desirable elements of a rest area facility:

1. Picnic Tables. Determine the desired number of picnic tables and ensure that 25% – 50% of the tables are sheltered. Sheltered tables are not required where an adequate number of trees would provide some shade for picnic tables. Sidewalks should interconnect approximately 50% of the picnic tables.

   The design of the shelters should be compatible with the design of the building facility. Generally, lighting in the shelters is not required, but certain locations may warrant some illumination. Anchor picnic tables to concrete pads to prevent vandalism. Where appropriate, place tables on parking lot islands or on the backside of truck parking areas.

2. Playground. Provide a children’s playground in a convenient location that attracts its use, but does not impede access to the building facility. Locate the playground area as near to the car parking area as practical, but a sufficient distance away to enhance the safety of playing children. A general size of 9 m x 12 m provides an acceptable area to install a variety of playground equipment.

   The area should have a fixed border to retain the surface material. Consider using a rubberised surface on top of concrete base for ease of maintenance and longevity of surface.

3. Pet Walk. Provide a pet walk area at all rest area facilities. The location should be a designated area that is well signed and well lit at night. In addition, the area should be away from the parking lot, picnic tables, and playground.

16.6.3.8 Utilities

Rest areas may be located in areas where utility services are not readily available. Water, electric, gas, and sewer services are necessary for a rest area facility. The following provides general information on these rest area utilities:

1. Water. It is highly desirable to obtain water service from an adjacent municipal system. If this option is not available, then use a well-water system. If a well system is used, provide for a secure storage area for water treatment chemicals and softener salt. Evaluate all water systems, well or municipal, for quality and, if deemed necessary, treat to remove objectionable elements.

2. Electricity. Obtain electrical service from the local utility company. Provide a high-efficiency, commercial-grade cooling unit in the facility to cool all areas of the building. Where feasible, include an emergency generator to operate enough lighting and machinery to function.

3. Gas. Consider natural gas where readily available to provide for the heat source. The use of propane gas is not desirable because of the potential for vandalism. Provide a high-efficiency, commercial-grade heating unit in the facility to heat all areas of the building and locate controlled access thermostats inside the mechanical room.

4. Sewer. It is highly desirable to connect sewer systems to a local municipal system. Where this is not practical, consider the construction of sewage lagoons with aeration capability.

16.6.3.9 Outdoor Electrical and Mechanical Equipment

Where practical, locate heating and air conditioning equipment indoors or on the building rooftop. If the only reasonable outdoor location is at ground level, then provide a security fence around the equipment that is architecturally compatible with the building design.

Locate electrical transformers and exterior electrical cabinets on an inconspicuous side of the building away from the view of entering patrons. If necessary, screen these items from view with plantings or architecturally compatible fencing.

16.6.3.10 Landscaping

When cost effective, preserve existing shade trees and other natural features to increase the aesthetic value of the site. Develop the facility with minimal disturbance to natural terrain and existing plant growth. Supplement existing vegetation with landscape treatments to achieve an environment conducive to rest and relaxation.

Promote the use of low-maintenance features in a facility's landscape plans. Consider providing earth mounding to screen features such as sewage lagoons and to enhance the visual effect of plantings. See the Abu Dhabi *Road Landscaping Manual* (52) for additional information on landscaping and plant material.

16.7 Oasis

At petrol stations, in addition to petrol, food, and drink, space is usually provided to allow trucks to park overnight. The parking area should be set back approximately 100 m outside of the normal right-of-way line. Entrance to the oasis should only be allowed from the main roadway. Provide fencing around the oasis to discourage entrance from the outside.

The truck parking area should typically provide a centre aisle 10 m wide with 45-degree diagonal parking on both sides; however, 35-degree may be used in confined areas. Design exit aisles to be 8.5 m in width, minimum, with an adjacent curbing. Design ramp widths and roadway geometry to accommodate a WB-20 design vehicle. Where WB-33D trucks are allowed, ensure the WB-33D design vehicle can be accommodated. Ramp widths are normally 5 m wide; however, increase this width to accommodate truck off tracking when providing a curved alignment. See Figure 16-12 for a typical oasis in Abu Dhabi.

Figure 16-12: Typical Oasis

16.8 Truck Lay-By

Along freeway, expressways, and truck routes, truck pull-off areas are provided to allow drivers of heavy vehicles to conduct short, purpose based stops including load checks, emergency stops, completing logbooks, and addressing associated operational needs. No services (e.g. rest rooms, vending machines) should be provided at these lay-by locations. The designer should consider the following:

- In rural areas, a truck lay-by should be provided approximately every 25 km (maximum).
- On single carriageway roads, provide a lay-by on both sides of the road so trucks do not have to cross in front of opposing traffic.
- The paved parking area should be separated from the main roadway shoulder with a 5 m to 6 m outside separation.
- The lay-by entrances, exits, parking spaces, and pavement design should be designed to accommodate a WB-20 design vehicle.
- Depending on available right-of-way, parking maybe either parallel or at 45 degrees.
- On truck routes, consider providing spaces for up to 50 trucks.
- Ensure sufficient sight distance and acceleration lengths are provided to allow the truck to safely merge back into traffic.
- Lay-bys must be physically separated from Main line traffic by a VRS/Guard Rail with proper end termination.
- See Figure 16-13 for typical truck lay-bys in Abu Dhabi.

16.9 Off-Street Parking

A proposed roadway project may incorporate some form of off-street parking. Typical applications may include:

- providing off-street parking to replace on-street parking which will be removed as part of a proposed project;
- the construction of a park-and-ride lot for commuters; or
- the construction of a new rest area or improvement to an existing rest area.

Figure 16-14 illustrates a typical off-street parking lot in Abu Dhabi.

Figure 16-13: Typical Truck Lay-Bys

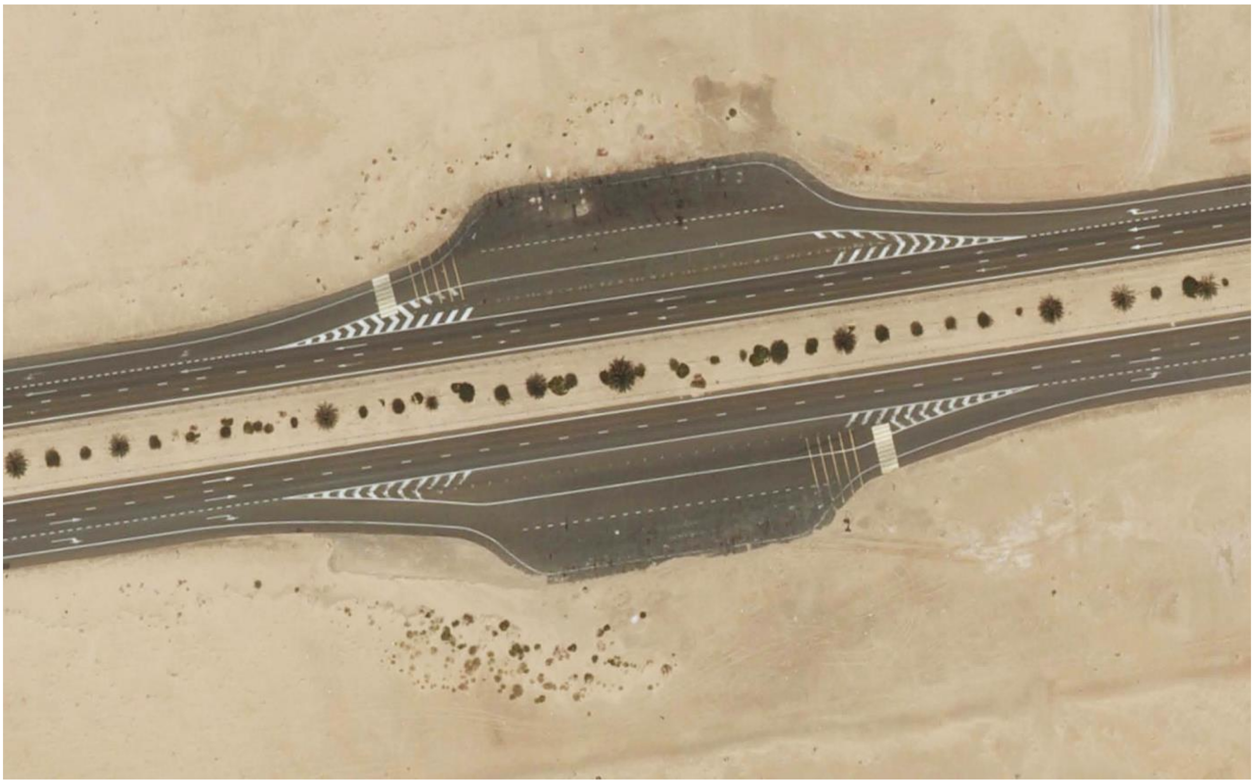


Figure 16-14: Typical Off-Street Parking (Liwa Street)

16.9.1 Park-and-Ride Lots

16.9.1.1 Location

Park-and-ride lots may be located in either rural or urban areas to accommodate car-pooling or to provide access to transit terminals. By locating these lots outside of urban areas, congestion is reduced, parking lot property costs are lowered, and accessibility is improved. The general location and size of park-and-ride lots is normally determined during the planning phase. Guidance for site selection can be found in the AASHTO *Guide for the Design of Park-and-Ride Facilities* (40). Some of the factors that will affect the location of the parking facility include:

1. <u>Site Availability</u>. Park-and-ride lots may consist of publicly owned property, excess right-of-way, or property used with the permission of private owners.
2. <u>Accessibility</u>. The lot should be convenient to residential areas, bus and rail transit routes, and major roadways used by commuters.
3. <u>Visibility</u>. The park-and-ride lot should be visible from the access road.
4. <u>Demand</u>. The lot must be large enough to accommodate the anticipated demand for parking spaces. In addition, sufficient transit service must be available to accommodate the anticipated demand.
5. <u>Congestion</u>. The location should precede any points of congestion on the major commuting roadway to maximise its benefits.

6. Design. The site location must be compatible with the design and construction of the lot. Considerations will include property costs, terrain, drainage, subgrade soil conditions, and available space in relation to the required lot size, visibility, and access.

7. Land Use. The location of the lot should be consistent with the present and future adjacent land use. Consider the lot's visual and other impacts on surrounding areas.

Consider the following when laying out a park-and-ride facility:

1. Entrances and Exits. Locate entrances and exits so that they have the least disruption to existing traffic, allow easy access to and from the lot, and provide the maximum storage space within the lot. In addition, consider the following:

    a. Location. Provide separated entrances and exits whenever practical, preferably on two or more roadways. The entrance should be on the upstream side of the traffic flow nearest the lot and the exit on the downstream side. If separation is not reasonable, the combined entry-exit point should be as close to mid-block as practical.

    b. Spacing. Separate entrances and exits should be at least 45 m apart and 45 m from an intersection. Desirably, these distances should be 100 m. For lots with less than 150 parking spaces, these dimensions may be reduced to 30 m.

    c. Traffic Signals. If a traffic signal is warranted or is expected in the future, the entrance should be more than 400 m from an adjacent traffic signal. Ensure that the traffic signal at the entrance can be interconnected and/or coordinated with the other traffic signals to allow vehicular progression along the route.

    d. Storage. Ensure that there is sufficient storage on the mainline for entering the lot. This may require providing separate left- and/or right-turn lanes. Also, check the exiting traffic to ensure that the exiting queue will not adversely affect the traffic circulation in the lot itself.

    e. Design. Design all entrances and exits for capacity, sight distance, turning radii, acceleration and deceleration lanes, turn lanes, etc., according to the criteria in Chapter 10 "Intersections." The typical design vehicle will be a CITY-BUS.

2. Drop-Off/Pick-Up Zones. Drop-off and pick-up zones for buses and autos should be clearly separated from each other and from parking areas to avoid as many internal traffic conflicts as possible. Circulation for drop-off/pick-up zones should be one-way and adjacent to the terminal loading/unloading area.

3. Traffic Circulation. Arrange the traffic circulation to provide maximum visibility and minimum conflict between small vehicles (e.g. autos, taxis) and large vehicles (e.g. large vans, buses). Locate major circulation routes at the perimeter of the lot to minimise vehicular-pedestrian conflicts. A counter-clockwise circulation of one-way traffic is preferred. This allows vehicles to unload from the right side.

4. Pedestrian and Cyclist Considerations. Consider pedestrian and cyclists routes when laying out the commuter lot. Avoid entrance and exit points in areas with high-pedestrian volumes,

if practical. Provide sidewalks between the parking areas and the modal transfer points. Maximum walking distances to any loading area should not exceed 300 m. Longer walking distances may require more than one loading area.

Crosswalks should be provided where necessary and should have the required pavement markings and signs (refer to the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22). Include signing and pavement markings for all pedestrian and cycle paths. In high-volume lots, fencing, barriers, or landscaping may be warranted to channel pedestrians and cyclists to appropriate crossing points. Crossings at major two-way traffic circulation lanes should have a refuge island separating the travel directions.

Include a cycle parking area relatively close to the loading area. If large volumes of cyclists are expected, provide a designated cycle lane to and from the cycle parking area.

16.9.2 Parking Design

To maximize the effective capacity of roadway improvements, sufficient off-street parking facilities should be provided to avoid the need for kerbline parking along primary roadways and main roads.

16.9.2.1 General

Parking facilities are of four general types:

1. Parking areas located parallel to, but physically separated from the main road.
2. On-street parking spaces, developed adjacent to the travelled lanes of local roads.
3. Independent parking lots developed off local roads.
4. Parking structures.

Each facility consists of an “aisle” area and a “standing area” (parking stalls). In the case of on-street parking, the moving lanes of the local road also serve as the aisle.

The following guidelines should be followed with regards to the design of parking facilities:

1. Minimum parking bay dimensions for perpendicular and angled parking is 5.5m x 2.7m. Parallel bays should be a minimum of 6.5m x 2.5m, although a length of 6.0m may be used if the parking bay is not adjacent to a main circulatory route.
2. Aisle widths for 2-way roads should be a minimum of 6.0m. Minimum aisle widths for 1-way roads vary dependent upon associated parking provision, from 4.0m (parallel and up to 45º angled parking), 4.5m for 60º angled parking and 6.0m for perpendicular parking.
3. For one-way roads, diagonal parking is preferred.
4. A sufficient number of parking spaces for disabled persons should be provided, based on the International Building Code (IBC). See Table 16-1 below, which is derived from the DMT.
5. Parking spaces for disabled persons should be located as close to building entrances and facilities as possible.
6. Wheel stops (100mm upstand) should be used to avoid cars overhanging walkways. These should be located at an offset as shown on the DMT Standard Drawings. This dimension

should be increased to 0.9m (forward parking) and to 1.1m (reverse parking) where structures (e.g. walls, substations, etc) or high kerbs are located close to the kerb edge.

7. Provide mid-block pedestrian crossing(s) for large parking areas.
8. Consider parking spaces for bicycles and motorcycles.
9. Consider garbage bin locations (in consultation with the Abu Dhabi Waste Management Department) to avoid improper placement of garbage bins in parking areas.

Reference should also be made to Department of Transport guidelines.

Table 16-1: Parking provision requirements for Disabled persons

Total Parking Bays Provided	Required Minimum Number of Bays for Disabled Persons
1 - 25	1
26 - 50	2
51 – 75	3
76 - 100	4
101 – 150	5
151 – 200	6
201 – 300	7
301 – 400	8
401 – 500	9
501 – 1,000	2% of total
1,001 and over	20, plus 1 No. for each 100 or fraction thereof over 1,000

Note:
(1) Based on guidelines provided in the International Building Code (IBC).
(2) Note the exceptions given in IBC Section 1106.

16.9.2.2 Parking Areas

The minimum safe distance from a main road intersection to a parking entrance or exit will be dependent on many factors, such as volume and speed of the traffic, type of intersection, width and number of lanes on the main road, the volume of traffic using the parking area and any sight distance restrictions.

Generally, it is desirable to locate parking exits onto main roads about 50m prior to the start of the left turn storage lane, and parking entrances off main roads about 60m prior to the intersection, and/or prior to the start of the free right turn taper.

The parking area edge nearest buildings should be set parallel to the building line and at a sufficient offset distance to allow inclusion of a sidewalk adjacent to the building.

16.9.2.3 On Street Parking Spaces

Parking spaces along local roads are provided immediately adjacent to the running lanes. These roads are generally 2-way roads and the associated parking should be either parallel or perpendicular.

The use of 45º and 60º parking should only be proposed on 1-way local roads.

16.9.2.4 Parking Lots

Parking lots are of two general varieties:

1. Single entrance/exit.
2. Double entrance/exit.

Wherever practical, the following layout rules should be applied:

1. Aisles and entrance/exit widths should be typically designed for two-way operation in conjunction with perpendicular parking.
2. Parking lots adjacent to each other (served from different aisles) should be separated by a raised sidewalk at least 1.0m wide which shall be unobstructed and clear from any signs or street lighting poles.
3. Diagonal parking should only be used in conjunction with 1-way aisles/local roads.

16.9.2.5 Parking Lot Dimensions.

See Figure 16-15. Parking stall dimensions vary with the angle at which the parking space is arranged relative to the aisle. From a traffic operations standpoint, one-way aisles are desirable and should be designed to provide counter clockwise circulation. When determining parking stall widths, consider the following:

- Typical stall widths (measured perpendicular to the parked vehicle) are 2.7 m.
- The minimum stall width for tandem parking and parking against the wall/obstruction is 2.7 m.

16.9.2.6 Parking Demand/Supply Analysis

During the early stages of the Concept Design, the designer should:

1. Determine the location of all existing parking facilities in the vicinity of the project.
2. Identify any facilities which will be displaced by the road improvements, which should therefore be replaced.
3. Determine the need for additional parking facilities and establish approximate locations for such parking.

The required analysis regarding parking will thus vary from project to project since parking demand is sensitive to site-specific factors, such as land use and proposed community developments.

Table 16-2 below indicates the parking requirements associated with different types of development. This provides a sample set of parking rates for a range of development types.

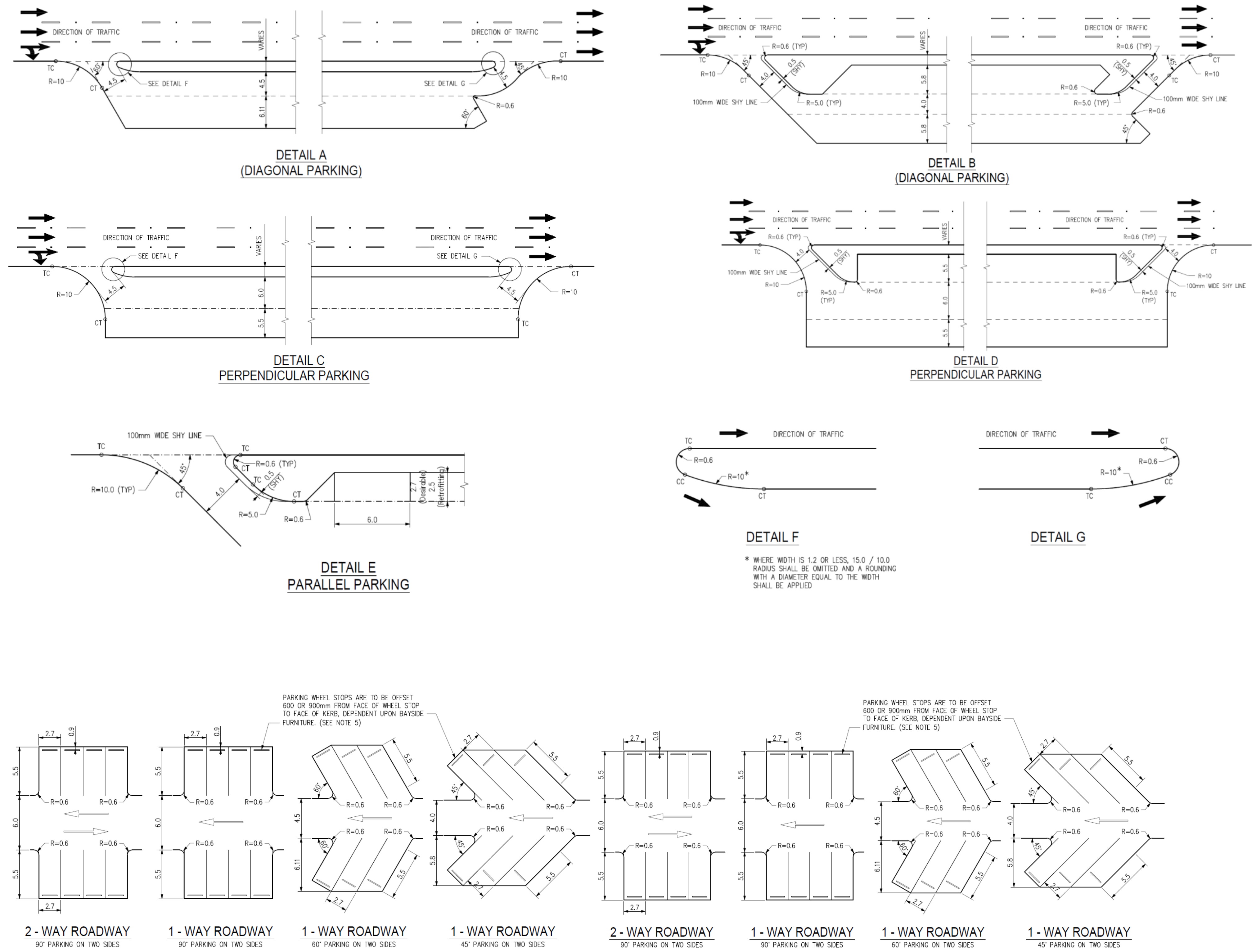
However, for full details of all categories, refer to the *"Trip Generation and Parking Rates Manual for the Emirate of Abu Dhabi"*, published by the Abu Dhabi DMT.

Table 16-2: Parking Requirements

Type of Development (Group Name)	Category Number Category Name	Class Number Class Name	Example of Parking Information (Type of Vehicle; Parking Rate)
Commercial Group	110 Shopping Malls / Centres	113-A Superstore	Car – Employees/Resident; 0.211 per 100m^2 of GFA. Car – Visitors; 1.704 per 100m^2 of GFA. School/Company Bus/Trucks; 0.015 per 100m^2 of GFA.
Office Group	210 Government Offices	211-A Local Government / Administrative Building	Car – Employees/Resident; 1.714 per 100m^2 of GFA. Car – Visitors; 0.252 per 100m^2 of GFA. School/Company Bus/Trucks; 0.016 per 100m^2 of GFA.
Residential Group	310 Apartments	311-A Studio and One Bedroom Apartments	Car – Employees/Resident; 0.818 per Unit. Car – Visitors; 0.043 per Unit. School/Company Bus/Trucks; 0.003 per Unit.
	320 Villa	321-A Standalone Villa	Car – Employees/Resident; 0.730 per Bedroom. Car – Visitors; 0.081 per Bedroom. School/Company Bus/Trucks; 0.000 per Bedroom.
	330 Group Accommodation	332 Labour Accommodation	Car – Employees/Resident; 0.178 per 100m^2 of GFA. Car – Visitors; 0.009 per 100m^2 of GFA. School/ Company Bus/Trucks; 0.115 per 100m^2 of GFA.
Institutional Group	510 Nursery and Schools	511 Nursery / Childcare	Car – Employees/Resident; 0.080 per Student. Car – Visitors; 0.392 per Student. School/Company Bus/Trucks; 0.017 per Student.
Medical Group	810 Hospitals	811 Government Hospital	Car – Employees/Resident; 0.281 per Bed. Car – Visitors; 1.066 per Bed. School/Company Bus/Trucks; 0.022 per Bed.

Notes:
(1) Parking rates quoted above apply to Abu Dhabi City – CBD, where applicable.
(2) These requirements should be considered as minimums.

Figure 16-15: Parking Lot Layout Dimensions



16.9.2.7 Lighting.

Desirably, the lot should be lighted for pedestrian safety and lot security. Ensure provisions are considered for lighting supports and power lines.

16.9.2.8 Shelters.

Pedestrian shelters are desirable when loading areas for buses and trains are provided. Their inclusion will be determined on a case-by-case basis.

16.9.2.9 Traffic Control Devices.

Provide signs and pavement markings to direct drivers and pedestrians to appropriate loading zones, parking areas, cycle facilities, parking, and entrances and exits. See the Abu Dhabi *Manual on Uniform Traffic Control Devices* (22).

16.9.2.10 Fencing. T

he need for fencing around a parking lot will be determined on a case-by-case basis.

16.9.2.11 Landscaping.

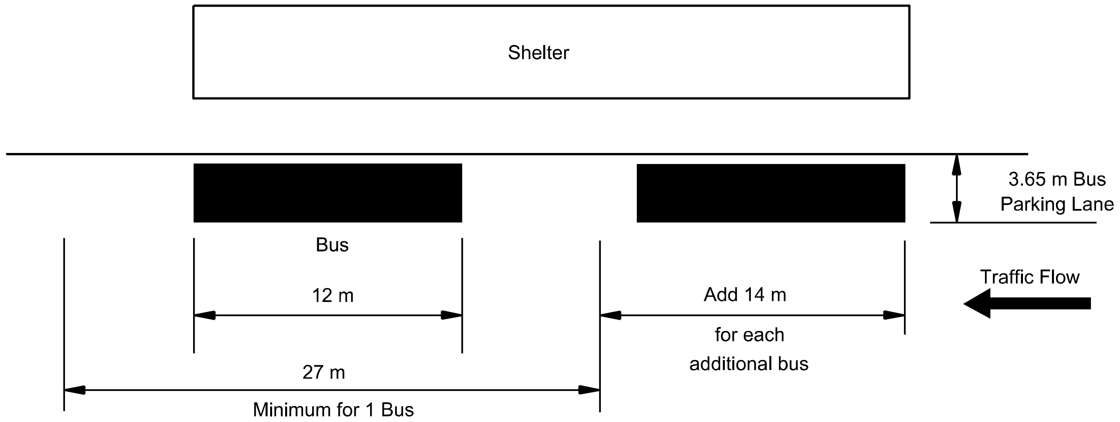
In some locations, consider landscaping to minimise the visual impact of the parking lot. This may include providing a buffer zone around the perimeter of the lot or improving the aesthetics of the lot itself. In addition, raised-curb islands and parking lot separators provide suitable locations for shrubs and trees. Landscaping should include low maintenance vegetation that does not cause visibility or security problems.

16.9.2.12 Bus Loading Areas.

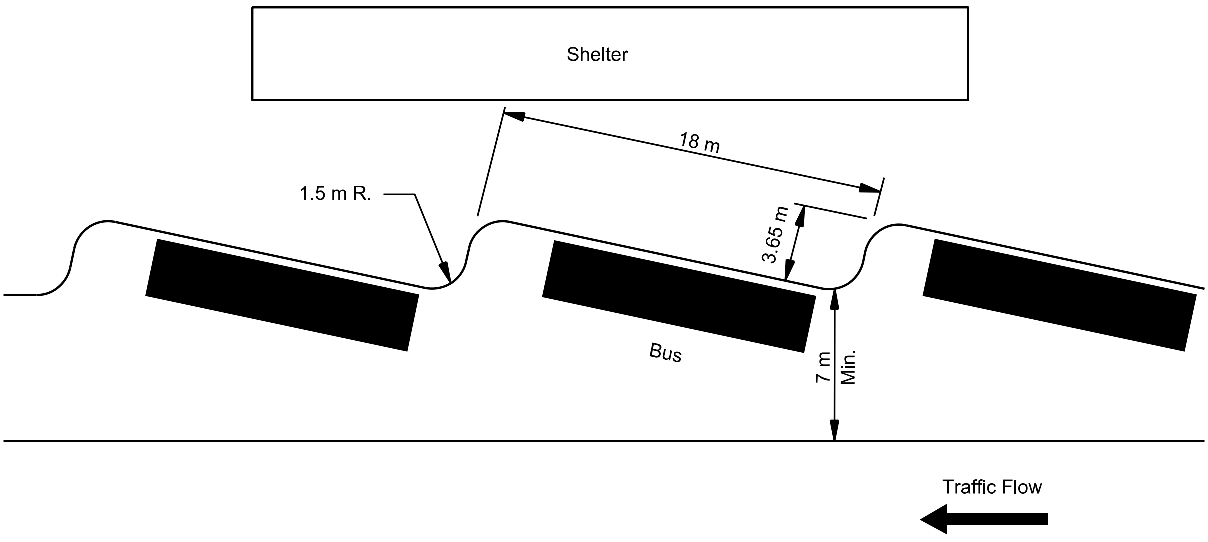
Bus loading and unloading areas located adjacent to park-and-ride lots should be designed to provide for continuous counter clockwise circulation and for curb parking without backing manoeuvres. The through traffic lanes and the curb loading area should each be a minimum of 3.65 m wide. See Figure 16-16.

See Public Transport Infrastructure *Design Basis – Bus Stations and Bus Depots* (49) for additional guidance.

Figure 16-16: Lengths for Bus-Loading Areas



Parallel Parking



Shallow Sawtooth Parking

CITED REFERENCES

Note: The documents issued by the former organizations UPC, DOT and DMAT are now under DMT

1. **AASHTO.** *A Policy on Geometric Design of Highways and Streets.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2011.

2. **Austroads.** *Guide to Road Design.* Sydney, Australia : s.n., 2010.

3. **Department of Transport.** *Design Manual for Roads and Bridges.* London, England : United Kingdom Department of Transport.

4. **DMT.** *Abu Dhabi Urban Street Design Manual.* Abu Dhabi : Abu Dhabi Urban Planning Council, 2012.

5. **AASHTO.** *A Guide for Achieving Flexibility in Highway Design.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2004.

6. **FHWA.** *Mitigation Stategies for Design Exceptions.* Washington, DC. : US Department of Transportation, 2007. FHWA-SA-07-001.

7. **NCHRP.** *NCHRP Report 480, A Guide to Best Practice for Achieving Context Sensitive Solutions.* Washington, D.C. : National Cooperative Highway Research Program, Transportation Research Board, 2002.

8. **ITE.** *An ITE Recommended Practice, Designing Walkable Urban Thoroughfares: A Context Sensitive Approach.* Washington, D.C. : Institute of Transportation Engineers, 2010. RP-036A.

9. **NCHRP.** *NCHRP Report 642, Quantifying the Benefits of Context Sensitive Solutions.* Washington, D.C. : National Cooperative Highway Research Program, Transportation Research Board, 2009.

10. **NCHRP** *NCHRP Synthesis 422 Trade-Off Considerations in Highway Geometric Design.* Washington, DC : Transporation Research Board, 2011.

11. **WSDOT.** *Understanding Flexibility in Transportation Design - Washington.* Olympia, Washington : Washington State Department of Transportation, 2005. WA-RD 638.1.

12. **DMT.** *ESTIDAMA website.* Abu Dhabi : Abu Dhabi Urban Planning Council, 2013.

13. **DMT** *Abu Dhabi Public Realm Design Manual.* Abu Dhabi : Abu Dhabi Urban Planning Council, 2012.

14. **FHWA.** *Transportation Planning of Sustainability Guidebook.* Washington, D.C. : Federal Highway Administration, January 2011.

15. **AASHTO.** *Center for Environmental Excellent by AASHTO website.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2011.

16. **Dubai Muncipality.** *Geometric Design Manual for Dubai Roads.* Dubai : Dubai Municpality, 1999.

17. **TRB.** *Highway Capacity Manual 2010.* Washington, D.C. : Transporation Research Board, National Research Council, 2010.

18. **DMT** *Transportation Impact Study Guidelines.* Abu Dhabi : Abu Dhabi Department of Transport, 2012.

19. **DMT** *Environmental, Health, and Safety Manual for Main Roads.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. DOT-M-08.

20. **DMT** *Environmental Assessment for Road Projects Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-537.

21. **DMT** *Access Management Policy and Procedures Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-521.

22. **DMT** *Manual on Uniform Traffic Control Device.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-525.

23. **DMT** *Walking and Cycling Master Plan.* Abu Dhabi : Department of Municipal Affairs and Transport.

24. **NCHRP.** *NCHRP Report 641, Guidance for the Design and Application of Shoulders and Centerline Rumble Strips.* Washington, D.C. : National Cooperative Highway Research Program, 2009.

25. **FHWA.** *http://safety.fhwa.dot.gov/roadway_dept/pavement/rumble_strips/.* Washington, D.C. : Federal Highway Administration (website), 2012.

26. **DMT** *Storm Water and Subsurface Drainage Manuals.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-513- to TR-516

27. **DMT** *Standard Drawings for Road Projects.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016.

28. **DMT** *Roadside Design Guide.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-531.

29. **DMT** *Road Structures Design Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-529.

30. **Kingdom of Saudi Arabia.** *Saudi Arabia Highway Design Manual.*

31. **DMT.** *Right of Way Utilities Distribution Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-520.

32. **DMT.** *Utility Corridors Design Manual.* Abu Dhabi : Urban Planning Council.

33. **AASHTO.** *A Policy on the Accommodation of Utililties within Freeway Right-of-Way.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2005.

34. —. *A Guide for Accommodating Utilities Within Highway Right-of-Way.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2005.

35. **FHWA.** *Manual on Uniform Traffic Control Devices for Streets and Highways.* Washington, D.C. : Federal Highway Administration, American Association of State Highway and Transportation Officials, Institue of Transportation Engineers, and American Traffic Safety Services Association, 2009.

36. **ITE.** *Freeway and Interchange: Geometric Design Handbook.* Washington, D.C. : Institute of Transportation Engineers, 2005.

37. **FHWA.** *Managaged Lanes: A Cross Cutting Study.* Washington, D.C. : Texas A&M University, College Station, TX for Federal Highway Administration, US Department of Transportation, November 2004. FHWA-HOP-05-037.

38. —. *Freeway Management and Operations Handbook.* Washington, D.C. : Federal Highway Administration, US Department of Transportation, September 2003. FHWA-OP-04-003.

39. **AASHTO.** *Guide for High-Occupancy Vehicle (HOV) Facilities.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2004.

40. —. *Guide for the Design of Park-and-Ride Facilities.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2004.

41. —. *Guidelines for Geometric Design of Very Low Volume Local Roads (ADT less than or equal to 400).* Washington, D.C. : American Association of State Highway and Transportation Officials, 2001.

42. **ITE.** *Urban Street: Geometric Design Handbook.* Washington, D.C. : Institute of Transportation Engineers, 2008.

43. **DMT** *Public Transport Bus Service Planning Guidelines.* Abu Dhabi : Department of Municipal Affairs and Transport, 2013

44. **NCHRP.** *NCHRP Report 395 Capacity and Operations Effects of Midblock Left-Turn Lanes.* Washington, D.C. : Transportation Research Board, National Research Council, 1997.

45. **DMT** *Traffic Signals and Electronic Warning and Information Systems Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-540.

46. **DMT** *Road Lighting Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-532.

47. **TRB.** *Transportation Research Circular 456 Driveway and Street Intersection Spacing.* Washington, D.C. : Transportation Research Board, National Research Council, March 1996.

48. —. Guidelines for Treatment of Right-Turn Movements on Rural Roads. *Transportation Research Record.* 1982, Vol. 855.

49. **DMT.** *Public Transport Infrastructure Design Basis - Bus Stations and Bus Depots.* Abu Dhabi : Department of Municipal Affairs and Transport, 2012.

50. **NCHRP.** *NCHRP Report 672 Roundabouts: An Informational Guide.* 2nd Edition. Washington, D.C. : Transportation Research Board, National Cooperation Highway Research Program, 2010.

51. **Highways Agency of Scotland Transport.** *Geometric Design of Roundabouts.* August 2007.

52. **DMT** *Road Landscaping Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016.

53. **AASHTO.** *User and Non-User Benefit Analysis for Highways.* Washington D.C. : American Association of State and Highway Transportation Officials, 2010.

54. **NCHRP.** *NCHRP Report 345 Single-Point Urban Interchange Design and Operational Analysis.* Washington, D.C. : National Cooperation Highway Research Program, Transportation Research Board, 1991.

55. **DMT.** *Pavement Design Manual.* Abu Dhabi : Department of Municipal Affairs and Transport, 2016. TR-527.

56. **NCHRP.** *NCHRP Report 505 Review of Truck Characteristics as Factors in Roadway Design.* Washington, D.C. : National Cooperation Highway Research Program, Transportation Research Board, 2003.

57. **TTI.** *Criteria for High Design Speed Facilities.* College Station, Texas : Texas Transportation Institute, 2007. FHWA/TX-07/0-5544-1.

58. **AASHTO.** *AASHTO Guide for the Development of Bicycle Facilities.* Washington, D.C. : Association of State Highway and Transportation Officials, 2012.

59. —. *Guide for Transportation Landscape and Environmental Design.* Washington, D.C. : American Association of State Highway and Transportation Officials, 1991.

60. —. *A Guide for the Development of Rest Areas on Major Arterials and Freeways.* Washington, D.C. : American Association of State Highway and Transportation Officials, 2001.

61. **DMT.** *Right-of-Way Manual.* Abu Dhabi : Department of Municipal Affairs and Transport.

OTHER REFERENCES

1. **DMT**. *Plan Abu Dhabi 2030: Urban Structure Framework Plan.* Abu Dhabi. Abu Dhabi Urban Planning Council.

2. **DMT**. *Plan Al Ain 2030: Urban Structure Framework Plan.* Abu Dhabi. Abu Dhabi Urban Planning Council.

3. **DMT**. *Plan Al Gharbia 2030: Urban Structure Framework Plan.* Abu Dhabi. Abu Dhabi Urban Planning Council.

4. **Abu Dhabi City Municipality**. *Roadway Design Manual – Roads and Bridges*. Abu Dhabi, 1998.

5. **Dubai Municipality**. *Geometric Design Manual for Dubai Roads*. Dubai.

6. **Department of Municipal Affairs and Transport (DMT).** Abu Dhabi Walking and Cycling Master Plan, Abu Dhabi

INDEX

A

B

C

D

E

F

G

H

I

J

L

S

T

U

V

W