“Highway Geometric Design Guide”, Chapter G, 3R/4R Geometric Design Guidelines | Transportation and Economic Corridors

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CHAPTER G
3R/4R GEOMETRIC DESIGN GUIDELINES

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G.1 INTRODUCTION

3R projects generally include resurfacing, restoration, or rehabilitation of existing paved roads. 4R projects include some reconstruction of existing paved roads, which generally takes place in conjunction with the resurfacing, restoration, or rehabilitation of the existing pavement. The purpose of the 3R/4R guidelines is to extend the service life of existing paved highways and enhance highway safety on a network basis. To accomplish this objective, the guidelines focus on the most cost-effective safety improvements and encourage the use of low-cost opportunities to improve safety where major reconstruction is not cost effective. The guidelines contained in this document are general in nature. All other parts of the “Highway Geometric Design Guide” (HGDG) are applicable, unless specifically noted here to be allowable exceptions.

The process used to review the geometric elements on existing paved highways is described below:

Newly constructed pavements are typically designed with a structural design life of 20 years. The structural design life of subsequent rehabilitations generally ranges from 10 to 20 years, however, available budget and actual pavement performance impact the pavement rehabilitation cycle. Generally, most pavements are expected to be rehabilitated every 15 to 30 years. This establishes a logical timetable for reviewing the geometric elements on existing paved roads. Required geometric improvements are generally most cost effective to construct at the time of pavement rehabilitation. Projects that are scheduled for pavement rehabilitation are listed on a construction program. The list is based on pavement condition and other considerations. An assessment of the geometric elements is made on each section of highway prior to pavement rehabilitation.

The general scope of work includes determining whether grade-widening is required and deciding if selective geometric improvements or general reconstruction (usually to new construction standards) is appropriate.

The initial review is based on a summary of geometric, traffic and collision data that is readily available for each project. Generally, as part of the initial review, projects will be identified for a planning study, preliminary engineering, or detailed engineering.

The Geometric Assessment Process Flowchart (Figure G-1a) shows an outline of the sequence of activities involved on a typical 3R/4R assessment.

The application of 3R/4R guidelines does not apply to bridge projects as structures have a longer design life. For bridge-specific requirements, refer to the Alberta Transportation, "Bridge Conceptual Design Guidelines" [1].

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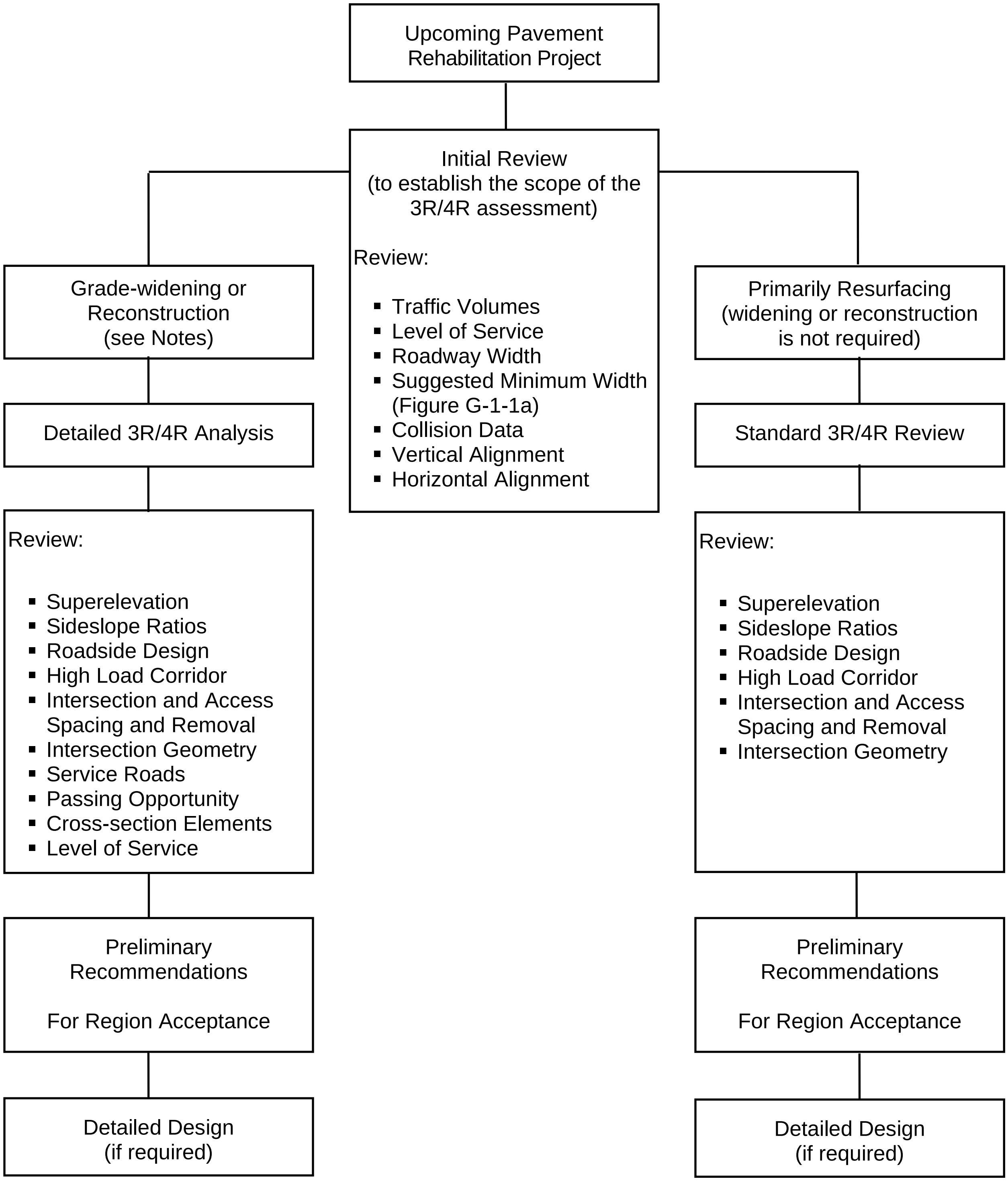
Figure G-1a Geometric Assessment Process Flowchart



Notes:

1. Reconstruction may only involve spot horizontal and/or vertical alignment improvement. Undertake benefit cost analysis as required.
2. Strategies to retain existing pavement width should be considered prior to grade-widening or reconstruction. Refer to Section G.1.3, Grade-widening/Reconstruction Versus Overlay and Chapter C.8.1, Strategies to Retain Existing Pavement Width.

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G.1.1 Guidelines for Initial Review

The initial review establishes the scope of the geometric assessment. The following information is required for the initial review: AADT, level of service, pavement width, collision data, summary of vertical alignment K values (for crest and sag curves), and summary of horizontal alignment information (including the radius of all curves).

When determining the scope of work for a 3R/4R project, a fundamental parameter that must be considered is pavement width. Refer to Section G.1.2, Minimum Lane and Shoulder Width, Section G.1.3, Grade-Widening/Reconstruction Versus Overlay, and Figure G-1-1a.

If grade-widening is warranted, an assessment of the horizontal alignment, vertical alignment, and collision data must be made to determine if simple grade-widening is required (i.e., retain the existing horizontal and vertical geometry) or some degree of reconstruction is required (e.g., horizontal and/or vertical alignment improvement). Refer to Section G.2, Horizontal Curvature, and Section G.3, Vertical Curvature.

If grade-widening is not required, the vertical alignment, horizontal alignment, and collision data are still reviewed to determine if selective alignment improvements are called for.

If neither grade-widening nor horizontal/vertical curve improvements are indicated by the initial review and there are no safety concerns, the project is primarily resurfacing and the standard 3R/4R review applies. Standard 3R/4R projects are further assessed for superelevation, sideslope ratio, roadside design (e.g., objects in the clear zone, barrier, etc.), high load corridor requirements, intersections, and accesses.

All other projects undergo a detailed 3R/4R analysis, which includes an assessment of the elements in a standard 3R/4R review as well as service roads, passing opportunity, cross-section elements (sideslope, ditch width and backslope) and level of service.

Like the initial review process for a rural highway, urban roads require a review of the existing conditions such as traffic volumes, level of service, pavement width, collision data, and roadway geometry. Since most of the urban highway network is through developed areas, additional constraints due to adjacent developments, accesses, and on-street parking, as well as pedestrian, bicycle and transit requirements are considered. Offsets to boulevard sidewalks, monolithic sidewalks, light poles, street furniture and other landscaping features are typically reviewed.

3R/4R projects in urban settings could range from minor improvements to a full assessment of safety. In the latter case, a review of traffic operations and a safety audit are likely needed to assess the cost versus benefits of possible improvements.

A benefit cost analysis is typically used to determine if improvements are justified in constrained areas. Refer to the Alberta Transportation, “Benefit Cost Model and User Guide” [2] for examples of economic analysis. Refer to the Alberta Transportation, “Design Exceptions Guideline” [3] if deviations from the HGDG are contemplated.

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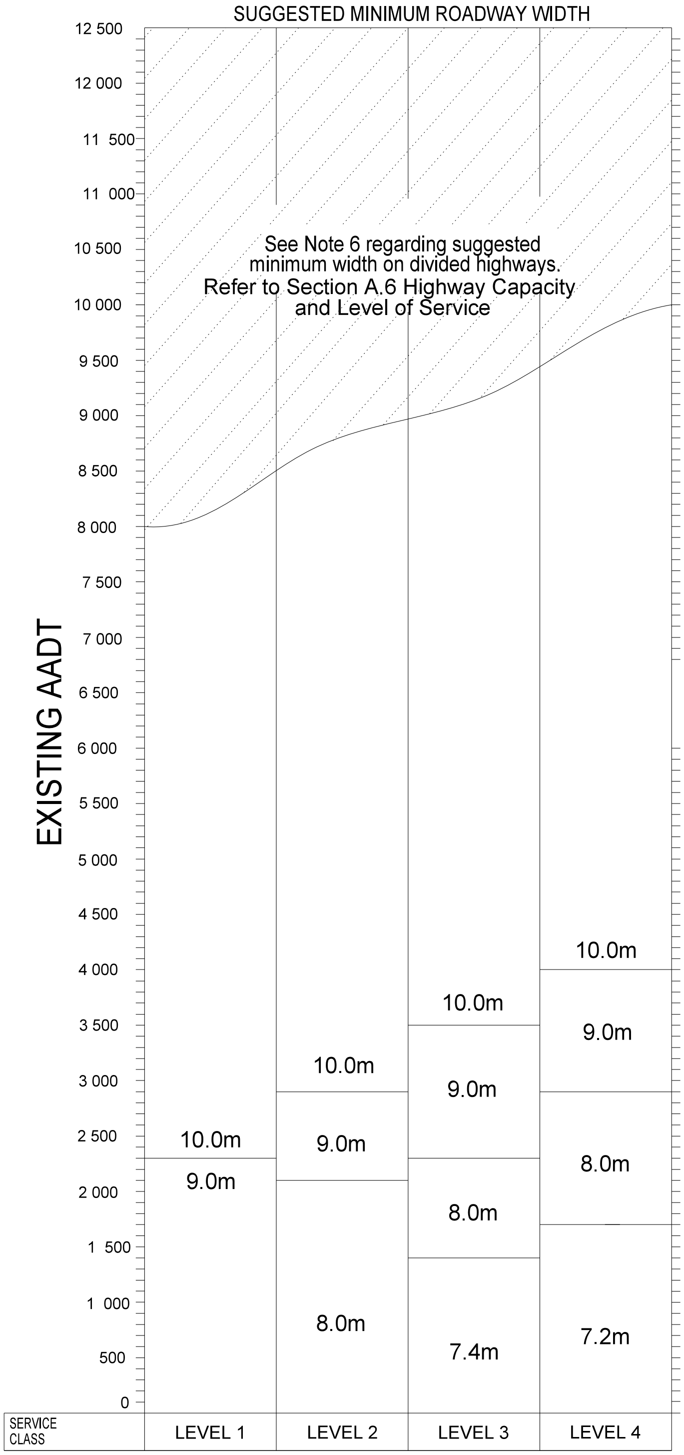
Figure G-1-1a Suggested Minimum Widths for Rural Two-lane Undivided Highways

(shown in terms of existing AADT)



NOTE: This chart has been developed as a guideline to assist in deciding whether a particular roadway, which requires pavement rehabilitation, should be widened. The chart should be used as follows:

1. A geometric assessment should be conducted on all pavement rehabilitation projects. The need for, and cost effectiveness of, grade widening will be considered as part of that assessment. This chart has been prepared to show cost effectiveness based on generalized assumptions for construction cost, collision rate and collision severity. The collision rates have been based on roadway width and traffic volume.

2. The chart shows suggested minimum widths for roadways of each classification, based on traffic volumes. The ranges have been established based on practical considerations, which limit the minimum widening that can be done, and the results of a life-cycle economic analysis, which includes safety benefits associated with grade widening. The existing AADT values have been obtained from the economic analysis, by determining the current traffic volume required to yield a satisfactory rate of return on construction investment, by the end of the design life. For this purpose, a 20 year design life and an average annual traffic growth rate of 2.0% (not compounded) was used. Generally, this is considered to be a good ballpark growth rate to apply to Alberta's rural roads based on examination of traffic growth patterns for the past 15 to 20 years. Where a more accurate 20 year traffic volume projection is available for a specific roadway, a designer may use that information to adjust the ranges shown here or run a special economic analysis to determine the cost effectiveness of grade widening on that particular project.

3. If the roadway after being overlaid would have a width less than that shown for the existing AADT and service class, then grade widening, reconstruction, or twinning is to be evaluated. Desirable width, as shown on Figure A-7-1A, should generally be used on all projects which involve a significant amount of grading work.

4. If the overlaid width is greater than, or equal to, the width shown and the collision history (rate and severity) is not of concern an overlay is appropiate .

5. Where reconstruction has been selected, construction to the new design standards is required. Deviation from those standards may be justified through the design exception process. The geometric and safety record of the existing roadway, the level of access control, the desirable design speed, the design AADT, the passing demand and opportunity should all be considered in that decision.

6. On existing twinned highways or highways that are being twinned the suggested minimum width on the existing two lane carriageway (roadway) is 9.5m.

REVISIONS	No. 2	By: XX		Date: Mmm dd/yy
	No. 1	By: XX		Date: Mmm dd/yy
Prepared: FS	Date: January 2023		Suggested Minimum Widths for Rural Two-Lane Undivided Highways (Existing AADT)	Figure G-1-1a
Checked: LM	Scale: NTS			

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G.1.2 Minimum Lane and Shoulder Width

G.1.2.1 Undivided Highways

Figure G-1-1a is used to determine if a given roadway will be sufficiently wide after an overlay, based on the existing AADT and service classification. If the roadway will not be sufficiently wide after an overlay, grade-widening and/or reconstruction are considered. Figure A-7-1a indicates the desirable width for the roadway, based on the design AADT, service classification and passing opportunity.

The suggested minimum widths in Figure G-1-1a are based on existing AADT. The desirable widths in Figure A-7-1a are based on design AADT.

G.1.2.2 Divided Highways

On existing divided highways that are being overlaid and existing undivided highways that are being twinned (i.e., to build a divided highway), the suggested minimum width for the existing roadway surface is 9.5 metres. This provides width for shoulders and lanes as follows: 0.3 m, 3.7 m, 3.7 m and 1.8 metres.

If the pavement width is greater than 9.9 m (0.3 m left shoulder and 2.2 m right shoulder), the width of the right shoulder is typically increased to 3.0 m before the left shoulder width is increased.

G.1.2.3 Ramps

On existing single-lane and dual-lane ramps, the typical minimum shoulder width is 0.3 m on the left and 1.8 m on the right. This is consistent with the 3R/4R guidelines for divided highways. Retaining the lane width(s) in accordance with the Alberta Transportation standard ramp lane width(s) is desired. This provides the following suggested minimum ramp widths:

- Single-lane ramps: 0.3 m, 4.8 m, 1.8 m = 6.9 m
- Dual-lane ramps: 0.3 m, 3.7 m, 3.7 m, 1.8 m = 9.5 m

A 6.9 m suggested minimum width on single-lane ramps should be considered where the horizontal radius is ≥ 90 metres. This is consistent with Table D-6-3-2 for Case II-C (SU-SU design vehicle). The 6.9 m minimum width allows an SU design vehicle to slowly pass a stalled SU design vehicle. Where the ramp radius is less than 90 m (e.g., a loop ramp), the suggested minimum ramp width is also based on Table D-6-3-2 for Case II-C (SU-SU).

If an existing ramp is frequently utilized by over-dimensional vehicles (e.g., on a high load corridor), the minimum ramp width should be evaluated on a site-specific basis, considering the number, width, and height of the loads.

G.1.2.4 High Load Corridors (HLC)

For any projects on the HLC, the designer must ensure the minimum overhead and lateral clearances are met. Refer to Chapter A.2.5.1, High Load Corridor, for further details.

Undivided Two-Lane Highways

When grade-widening or reconstruction is warranted on existing and proposed high load corridors, selecting a roadway width of 10 metres or greater should be considered. Refer to Figure A-7-1a.

On high load corridors, when the roadway width, after overlay, is less than 10.0 metres and the width meets the suggested minimum width indicated in Figure G-1-1a, widening may still be warranted.

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A 10-metre-wide road provides room for a 7.3 m wide load (the maximum oversize dimension for a divisible load) to pass a 2.6 m wide load (the maximum legal size) at a reduced speed. Refer to Chapter A.2.5.1, High Load Corridor, and Figure A-2-5-1a for more information.

An economic analysis that considers the number, width, and height of the loads as well as escort measures may be required to determine if the wider roadway is justified. The analysis should also consider site-specific factors that include traffic volume, traffic composition, existing and/or proposed truck staging areas, safety rest areas, climbing/passing lanes, and intersections.

Information on the number of loads (including input on future loads), widths, heights, etc. of permitted oversized loads may be obtained through the TRAVIS database from the Alberta Transportation, Permitting and Approvals Section.

Divided Highways

On an existing divided highway (four lanes) that is being overlaid, or an undivided highway that is being twinned, the suggested minimum width for each roadway surface is 9.5 metres. Refer to Note 6 in Figure G-1-1a. The rationale for the suggested minimum width on a divided highway (9.5 m) being less than on an undivided highway (10.0 m) are as follows:

- The collision risk is reduced because the vehicles are travelling in the same direction;
- Due to the presence of interchanges and/or intersections, there may be frequent opportunities for an oversize vehicle to pull over and allow following vehicles to pass;
- Regular traffic typically has no need to stop or back-up; and
- Delay and frustration are reduced due to the one-way operation of the divided highway.

G.1.3 Grade-Widening/Reconstruction Versus Overlay

Refer to Chapter C.8.1 for details on the Alberta Transportation strategy to retain existing pavement width. When pavement rehabilitation is being undertaken, all possible alternatives to minimize the loss of surface width are to be considered. Although the objective is to minimize loss of pavement width on a network basis, preventing pavement loss on all projects is impractical. Inevitably, there will be a need to widen or reconstruct some roadways. When choosing between full grade-widening, partial grade-widening and total reconstruction, many factors must be considered. The suitability of various options will depend on structural needs, existing conditions, and funding availability. Normally, input will be required from Alberta Transportation before a decision is made. The following guidelines may be useful:

- If the width after overlay will be less than that shown in Figure G-1-1a, grade-widening is generally cost effective based on reduction of collision costs, assuming a provincial average collision rate for the existing width of roadway.
- Compare the existing collision rate to the provincial benchmark value, for the roadway width, in the Network Expansion System Support (NESS) reports. A collision rate that is significantly different from the benchmark value should be considered when assessing the need for grade-widening. When assessing the collision history of a particular road, the breakdown of collision types and their relationship to geometric features are considered. For example, geometric improvements will usually have little impact on the number of animal collisions. Roadside improvements can significantly reduce the severity of run-off-road collisions.
- Existing geometry that does not meet 3R/4R criteria may provide more support for grade-widening or reconstruction than an overlay.
- Where grade-widening and horizontal or vertical alignment improvements are warranted, a designer should assess the impact of only doing selective alignment improvements. Selective alignment improvements will likely be less costly than applying the desirable new construction standards throughout; however, they may result in a less balanced design (e.g., a wide roadway with minimum alignment standards). Vehicle speed generally increases because of lane and

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shoulder width improvements. The speed increase will offset part of the safety benefit of grade-widening because, other things being equal, collision rates and severity increase with speed. Because the typical driver expects a better alignment on a wider road and drives accordingly, it is appropriate to provide better than minimum alignment standards on roadways with wide shoulders.

Where a grade-widening project requires alignment improvements over a substantial portion of its length, it is appropriate to adopt the desirable new construction standards for the entire project to ensure design consistency.

G.1.4 Acceptable Curb Heights on Existing Roadways

The acceptable curb height on an existing roadway (after overlay) is dependent on the curb location, considering pedestrian safety. Table G-1-4a provides a general guideline for selecting minimum curb height.

Table G-1-4a Minimum Acceptable Curb Height on Existing Roadways

Curb Location	Minimum Acceptable Curb Height
Adjacent to sidewalk	75 mm
Adjacent to boulevard	50 mm
On median side	50 mm

G.2 HORIZONTAL CURVATURE

On pavement rehabilitation projects, improvements are considered on all horizontal curves. Generally, improvements are warranted on curves that do not meet the minimum radius for new construction. Alberta Transportation uses a maximum superelevation rate of 0.06 m/m for all rural roads. The minimum radius for each design speed is indicated in Table B-3-5-3a. Curves that exceed the minimum radius should also be considered for upgrading based on factors such as superelevation rate, collision rate, intersections on curve, hazards on curve, highway alignment consistency, coordination of horizontal and vertical alignments, or road user savings due to lower vehicle running costs.

Generally, a summary of all horizontal alignment elements is prepared. The summary includes the curve geometric information (radius, spiral parameters, deflection angle, superelevation, width, length, and grade), collision records, and traffic information (e.g., AADT, ASDT, DHV, vehicle composition).

Where detailed analysis is required, use of the Alberta Transportation, “Benefit Cost Model and User Guide” [2] is recommended as one tool in the analysis.

Because of the many site-specific factors that can affect the outcome of an analysis, a project-specific analysis considering all the alignment alternatives should be undertaken. When the realignment proposals involve more than one curve on a highway section, the entire alignment is included in the analysis (from common point to common point, which includes all the alternatives). The route to be used should include all expected costs that apply to the specific project.

The “Benefit Cost Model and User Guide” includes benefits for reductions in vehicle running cost, reductions in travel time for shorter alignments or higher speeds, and potential savings in collision costs. Three factors may contribute to savings in collision costs for horizontal alignment improvements. These factors are:

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- Alignment Length may be different for different alternatives. This will result in differences in total vehicle kilometres and, therefore, differences in exposure of vehicles to collision risk over the analysis period. The alignment length also affects vehicle running cost and travel time.

- Collision Severity on any project could be affected by horizontal alignment improvement. Presently, there is no available data to link these two factors. Generally, where horizontal realignment is being considered, if all other geometric features are to remain unchanged, no change in collision severity should be assumed. The analysis should consider that other improvements (e.g., sideslope flattening or removal of obstacles such as approaches), which are often undertaken at the same time as horizontal realignment, could result in lower collision severity. In this case, a reduction in collision severity may be assumed.

- Collision Rate (generally expressed as collisions per 100 million vehicle kilometres) is known to be related to the deflection angle and radius of horizontal curves. Many models have been developed to predict the collision rate on horizontal curves and tangents of high-speed rural highways. Refer to the “Crash Modification Factors Clearinghouse” website [4] for more information. An example showing the benefit cost analysis for horizontal alignment improvement is included in the training materials of the Alberta Transportation, “Benefit Cost Model and User Guide” [2].

On many projects where horizontal realignment is considered, cost effectiveness is demonstrated without a significant saving in collision costs. On some projects, the collision cost savings are crucial to the overall cost effectiveness. In these cases, a sensitivity analysis should be performed (i.e., an analysis which shows the cost effectiveness based on a range of collision rates that may result after geometric improvement). The results of the sensitivity analysis will better inform the decision regarding realignment based on a reasonable prediction of the change of collision rate and collision severity.

When horizontal realignment of existing paved roads takes place, the new alignment is based on new construction standards. Minimum design values are typically reserved for circumstances at critical locations, with non-minimum values being used in the majority of cases where practical and cost effective. Non-minimum values result in lower superelevation rates, less wear and tear for vehicles and tires on curves, and generally safer and more relaxed driving conditions for all road users.

Available right-of-way and adjacent developments may restrict implementation of potential geometric improvements. A benefit cost analysis may determine if improvements are justified. Refer to the Alberta Transportation, “Benefit Cost Model and User Guide” [2]. Refer to the Alberta Transportation, “Design Exceptions Guideline” [3] if deviations from the HGDG are contemplated.

G.3 VERTICAL CURVATURE

On pavement rehabilitation projects, improvements are considered on all vertical curves.

G.3.1 Crests

Reconstruction of vertical crest curves is considered if any one of the following conditions exist:

1. A safety concern; or
2. A hazard (or hazard potential such as an intersection) near the crest; or
3. Limited sight distance (i.e., the available stopping sight distance is substantially less than that required for new construction for vehicles travelling at the design speed).

Although reconstruction of vertical crest curves may be appropriate where any of the above conditions exist, reconstruction may not be the most desirable action due to cost effectiveness, physical constraints, or other reasons.

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The use of traffic control devices to advise motorists of the sharp crest curve may be considered when a decision has been made to not improve a vertical alignment due to low cost-effectiveness. Additional measures to be considered are warning signs, fixed hazard removal, shoulder widening, and relocation of minor intersections.

Generally, a substantial sight distance restriction is one where the available stopping sight distance is less than that required for a speed that is 20 km/h less than the design speed. In Alberta, the design speed for rural two-lane undivided highways is typically 110 km/h. Therefore, the suggested minimum vertical crest curve on those roadways is generally based on the minimum stopping sight distance for 90 km/h.

As indicated in Table B-2-3a, the minimum stopping sight distance is 160 m (on a level grade) for a design speed of 90 km/h. The corresponding minimum K value for a vertical crest curve is 39 (Table B-4-4-2a). Therefore, a K value of 39 is considered acceptable on an existing paved undivided highway with a design speed up to 110 km/h.

Table G-3-1a shows suggested minimum K values for vertical crest curves (for 3R/4R projects) based on the above criteria.

Table G-3-1a Minimum K Value for Vertical Crest Curves (for 3R/4R Projects)

Design Speed (km/h) (see Note 3)	Speed Used for Minimum Vertical Crest Curve (km/h)	Minimum Stopping Sight Distance (SSD) (m)	Minimum K Value (if SSD < LVC)	Minimum K Value (if SSD > LVC)
130	110	220	74	Depends on A (see Note 3)
120	100	185	52	
110	90	160	39	
100	80	130	26	
90	70	105	17	
80	60	85	11	
70	50	65	7	
60	40	50	4	

Notes:
1. LVC is the horizontal length of the vertical curve (m).
2. A is the algebraic difference in gradient between the two intersecting gradelines (%).
3. Where the design speed is different from that shown in Table G-3-1a, or where the required (minimum) stopping sight distance exceeds the length of the vertical curve (SSD > LVC), the formulae in Figure B-4-4-2a are used to determine the minimum K value for the crest vertical curve.

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G.3.2 Sags

Consideration for the improvement of existing vertical sag curves is typically based on comfort control, rather than headlight control. Refer to Chapter B.4.4.3, Sag Vertical Curves, for information related to comfort control and headlight control. Table B-4-4-2a indicates the minimum K values for sag vertical curves.

Table G-3-2a shows suggested minimum K values for vertical sag curves (for 3R/4R projects) based on the above criteria.

Table G-3-2a Minimum K Value for Vertical Sag Curves (for 3R/4R Projects)

Design Speed (km/h)	Minimum K Value
130	44
120	37
110	32
100	26
90	21
80	17
70	13
60	10

Notes:

1. Where the design speed is different from that shown in Table G-3-2a, the formula (for comfort control) in Figure B-4-4-3a is used to determine the minimum K value for the sag vertical curve.

G.3.3 Vertical Curvature Improvement

When vertical realignment of existing paved roads takes place, the new alignment is based on new construction standards. Minimum design values are typically reserved for circumstances at critical locations, with non-minimum values being used in the majority of cases where practical and cost effective. Non-minimum values result in better passing opportunity, better-than-minimum sight distance (e.g., stopping, decision and intersection), and generally safer and more relaxed driving conditions for all road users.

Available right-of-way and adjacent developments may restrict implementation of potential geometric improvements. A benefit cost analysis may determine if improvements are justified. Refer to the Alberta Transportation, “Benefit Cost Model and User Guide” [2]. Refer to the Alberta Transportation, “Design Exceptions Guideline” [3] if deviations from the HGDG are contemplated.

G.4 INTERSECTIONS

Annual collision statistics compiled over years in Alberta indicate around half of rural intersection right angle and left-turn-across-path collisions resulted in fatality or injury. After head on collisions, they are the second and third most severe collision type.

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Chapter D, At-grade Intersections, includes information to review the functional, geometric, and operational aspects of existing intersections, including layout, nearby accesses, intersection sight distance, gradient, superelevation, and capacity. In urban settings, intersection analysis is often complicated by the existence of traffic control signals, the adequacy of which will also require assessment. Intersections in urban areas are often constrained by adjacent development, restricted right-of-way, and other surface features often not prevalent in rural settings.

To provide safe operation, intersections are reviewed to ensure they accommodate all vehicles that use them on a regular basis. The available intersection sight distance (ISD) in both directions, for each vehicle type, is measured on site using the eye height and object height shown in Table G-4a. Alternatively, the available ISD may be determined from the centreline profile if an accurate as-built profile is available and there are no horizontal sight line restrictions. The collision history at each intersection is also reviewed before deciding if corrective measures are appropriate.

The available ISD is compared to the required ISD for each design vehicle, based on the posted speed of the roadway. Refer to Chapter D.4, Sight Distance at Intersections.

The required intersection sight distance is based on providing sufficient sight distance (for the crossing driver) so the design vehicle, having come to a stop on the intersecting road, can safely make a left turn onto the highway without being struck by a vehicle approaching from the left. For new construction, the vehicle on the highway is assumed to be approaching at the design speed. For 3R/4R evaluations, the posted speed is used.

The required sight distance, based on posted speed, is generally acceptable at an existing intersection. Longer sight distances, based on design speed, are generally required at newly constructed intersections and existing intersections that are being reconstructed. Designers must assess the turning movements and vehicle composition at an intersection to ensure an adequate intersection sight distance is provided.

Table G-4a Minimum Sight Distance for At-grade Intersections (for 3R/4R Projects)

Design Vehicle (see Note 1)	Eye Height (m)	Object Height (m)	Minimum Acceptable Sight Distance for Left Turn onto Highway (m)					
			Based on Posted Speed (km/h)					
			60	70	80	90	100	110
WB-21 WB-23	2.3	1.3	310	360	410	460	510	565
WB-15 WB-17	2.3	1.3	235	275	315	350	390	430
SU (including bus)	1.8	1.3	180	210	235	270	295	330
Passenger Vehicle (P)	1.08	1.3	115	135	155	175	195	215

Notes:

1. Refer to Figure D-4.2.2.2 in Chapter D for sight distance requirements for other vehicles.
2. The usefulness of providing an intersection sight distance more than 500 metres has been debated. Changes to this table may be made based on research into gap acceptance by large trucks on rural highways in Alberta.

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G.5 PASSING OPPORTUNITY (CLIMBING, PASSING LANES)

Pavement rehabilitation projects should be evaluated for passing opportunity. On a highway segment, the available passing opportunity due to pavement markings, together with the passing demand (which is a function of traffic volume and speed distribution), will have a major impact on the level of service. Refer to the warrants and guidelines for climbing and passing lanes contained in Chapter B to determine the need for auxiliary lanes.

When applying the auxiliary lane warrants to existing paved roadways, there may be cases where the addition of an auxiliary lane is not desirable, even though the warrant is met. This could be due to the physical constraints of the existing roadway, presence of hazards such as intersections, or the location of no passing zones. For safety reasons on some projects, it may be necessary to include some access control in conjunction with the construction of auxiliary lanes. When this is not feasible, it may be better not to build the auxiliary lane.

For two-lane undivided roadways in an urban setting, passing is usually not allowed due to geometric parameters, an increased number of access points, on-street parking and the possibility of pedestrians and cyclists. Capacity of the highway is a function of the number of lanes, lane geometry, traffic signal timing, pedestrians, and on-street parking.

Refer to Chapter B, Alignment Elements, for more information.

G.6 ROADSIDE DESIGN

G.6.1 Clear Zone

Clear zone is a generally clear traversable area, adjacent to the edge of the travelled way, to accommodate the occasional errant vehicle that enters the roadside. The clear zone distance is a function of the design speed, roadside geometry, and traffic volume. Generally, any hazard located within the clear zone distance should be mitigated.

Refer to the Alberta Transportation, "Roadside Design Guide" [5] for more details on the clear zone concept and hazards that should be considered for mitigation.

G.6.2 Rumble Strips

Centreline rumble strips, shoulder rumble strips, and transverse rumble strips are to be installed on all roadways at the time of new construction, grade-widening, and pavement rehabilitation (overlay).

The purpose of shoulder and centreline rumble strips is to alert drivers when they inadvertently leave the travel lane. Rumble strips are not permitted on bridge decks due to increased water ponding, which leads to premature deck deterioration.

Transverse rumble strips are to alert drivers to a stop condition at an upcoming intersection. The current practice of Alberta Transportation is to install transverse rumble strips in advance of all stop-controlled intersections (on the stop-controlled approaches) in rural areas where the posted speed limit is 80 km/h or greater on the stop-controlled roadways, and the highway and intersecting approaches are paved (and under provincial jurisdiction).

Refer to Chapter H.4.9, Rumble Strips, of the Alberta Transportation, “Roadside Design Guide” [5], and Design Bulletin 18, Rumble Strip Placement Practices [6] for more information.

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G.6.3 Improvement to Sideslope, Ditch and Backslope

Existing sideslopes steeper than 4:1 are evaluated as candidate locations for improvement. Refer to the Alberta Transportation, "Roadside Design Guide" [5] for barrier warrant details if the sideslope cannot be flattened. On projects where it is not cost effective to improve the sideslope on the entire project (e.g., due to low traffic volumes), improvements should be considered at locations where run-off-road collisions are likely to occur (e.g., on the outside of smaller radius horizontal curves). On existing paved roads, sideslopes of 4:1 or flatter may not warrant improvement.

Where sideslope improvement is undertaken, a 4:1 slope is considered a minimum. A 5:1 slope is desirable for moderate volumes (design AADT 1500 – 4000) and a 6:1 slope is desirable for higher volume two-lane roadways (design AADT > 4000) and all divided highways. When the existing ditch width is reduced to accommodate sideslope improvement, 1.2 m is considered the minimum width. The ditch is to be sufficiently wide to provide adequate drainage and snow storage capacity.

Although backslopes are not as critical as sideslopes for an errant vehicle, providing 3:1 or flatter backslopes improves the ability of a vehicle to traverse the entire roadside cross-section.

The traffic volume ranges in Table G-6-3a are a guide for determining the extent of sideslope improvement that is warranted at the time of pavement rehabilitation. These ranges are based on an economic analysis of the cost effectiveness of sideslope improvements and consideration of the new construction standards for roadways with these volumes.

Table G-6-3a Sideslope Improvement Warrants

Design AADT	Sideslope Improvements Warranted	Assumptions
0 – 200	Improve sideslope to 4:1 (or flatter) at locations with run-off-road collisions.	Existing sideslope is steeper than 4:1.
200 – 300		
Pavement Width ≤ 8.2m	Improve sideslope to 4:1 (or flatter).	
Pavement Width > 8.2m	Improve sideslope to 4:1 (or flatter) at locations with run-off-road collisions.	
300 – 1500	Improve sideslope to 4:1 (or flatter).	
1500 – 4000	Improve sideslope to 4:1 (minimum) or 5:1 (desirable).	
> 4000 (undivided) and All Divided Highways	Improve sideslope to 4:1 (minimum) or 6:1 (desirable).	

Notes:

1. Refer to the Alberta Transportation, "Roadside Design Guide" [5] for barrier warrants if the sideslope cannot be improved to 4:1 or flatter.

G.6.4 Access Management and Treatment of Approaches

Typically, access management is considered prior to all overlay, rehabilitation or widening projects. Existing approaches are assessed for possible elimination, consolidation, or improvement.

At an early stage in the evaluation process, the designer and the project sponsor should examine the pertinent factors affecting the access management and agree on a strategy to determine the need for, and scheduling of, improvements to existing accesses.

Classification: Public

The degree of access management to be applied depends on a number of factors, including:

- type of construction;
- existing and future functional classification;
- access type (public or private) and spacing;
- intersection geometry (e.g., intersection treatment, sight distance, angle of intersection, etc.);
- traffic characteristics (e.g., AADT, ASDT, DHV, heavy vehicle composition, etc.);
- collision record (e.g., frequency of run-off-road collisions, angle collisions, or other collisions related to access);
- level of service;
- design speed; and
- ultimate design considerations (long term plan).

Ultimately, all private direct accesses to provincial highways are to be removed and redirected to public roads. When determining the number, location and spacing of approaches to adjacent properties or roadways serving adjacent lands, refer to Chapter I, Access Management Guidelines. These guidelines are typically used as basic requirements on new construction and reconstruction projects and may be used as a basis for access management on pavement rehabilitation projects.

Designers should review strategies to remove or combine existing highway accesses. When accesses are to be retained, the following guidelines are used:

1. To determine the need for improvements to existing accesses, a review of access management and access geometry should be undertaken early in the evaluation process. Minor access improvements often occur in conjunction with pavement rehabilitation. Major access management initiatives are normally undertaken as part of major upgrading projects (i.e., projects with a significant grading component).
2. Discuss any upcoming municipal development with the municipality and provide an opportunity for coordinating municipal-funded intersection improvements.
3. The sideslope of an approach is a key factor affecting safety. The slope is generally variable due to the transition from highway embankment to approach embankment. The slope located midway between the highway shoulder and basic right-of-way boundary is used as a criterion. Existing approach sideslopes that are steeper than the acceptable slope indicated in Table G-6-4a should be improved to the appropriate sideslope indicated in Figure D-3.3a (for a main intersecting roadway) or Figure D-3.3b (for a minor intersecting roadway). The sideslopes indicated in Figures D-3.3a and D-3.3b also apply when a new approach is installed or when the existing culvert in an approach requires replacement.
4. Where a culvert is required on a new approach, or where culvert replacement is required in an existing approach, the culvert is typically placed as far away from the highway as possible while still accommodating the ditch drainage. Placement near the highway right-of-way boundary is desirable. Refer to Chapter C.4.6, Culvert Installation, as well as the appropriate drawings in the Alberta Transportation website, “CB6 Highway Standard Plates – Active” [7] for design and installation parameters.
5. The geometry of other embankments within, or close to the clear zone of a highway right-of-way (e.g., railway embankments or irrigation canal embankments) are assessed in a similar way to roadway approaches. Because of the additional hazard posed by irrigation canals, there is a greater need to provide protection for the motorist and, therefore, relocation of accesses that are near canal crossings may be justified. The purpose of the relocation is to allow appropriate traffic barriers to be placed to protect highway traffic from the canal hazard and to allow for gentle slopes.
6. Under special or site-specific circumstances, a customized design may be appropriate rather than attempting to apply a typical solution that is impractical. Judgment is often based on knowledge of the safety and geometric information of the highway near the existing access (e.g., collision history, geometric parameters, presence of bridges or irrigation canals, aesthetics).

Classification: Public

7. Accesses on the outside of horizontal curves, where run-off-road incidents are more likely to occur, should be given a high priority for improvement.
8. At the time of pavement rehabilitation, all intersecting roadways, farm accesses and field accesses are assessed and appropriately surfaced. Refer to Standard Drawings CB6-3.50M2, CB6-3.50M3 and CB6-3.50M3a [7] for information related to the paving limits of intersecting roadways, farm (residential) accesses and field accesses.

Table G-6-4a Acceptable and Required Approach Sideslopes

Existing AADT	Fill Height	Steepest Acceptable Slope on an Existing Approach (see Note 1)		Typical Slope on a New or Improved Approach
		Projects with Minimum Grading Component (see Note 2)	Projects with Major Grading Component (see Note 3)	
Undivided Highway AADT < 1,000	< 4 m	3:1	4:1	Refer to Figure D-3.3a and Figure D-3.3b
	≥ 4 m	3:1	3:1	
Undivided Highway 1,000 < AADT < 3,000	< 4 m	3:1	5:1	
	≥ 4 m	3:1	3:1	
Undivided Highway AADT > 3,000	< 4 m	4:1	5:1	
	≥ 4 m	3:1	4:1	
Divided Highway AADT < 6,000	< 4 m	4:1	5:1	
	≥ 4 m	3:1	4:1	
Divided Highway 6,000 < AADT < 15,000	< 4 m	4:1	6:1	
	≥ 4 m	3:1	5:1	
Divided Highway AADT > 15,000	< 4 m	6:1	7:1	
	≥ 4 m	4:1	5:1	

Notes:
1. The approach slope is measured at a point midway between the highway shoulder and the basic right-of-way boundary.
2. Projects with minimal grading may include pavement rehabilitation projects and projects with isolated grading work (e.g., intersection improvement).
3. Projects with a major grading component include sideslope improvement, grade-widening, and reconstruction.

Classification: Public

G.6.5 Barriers and End Treatments

All projects are evaluated for hazards in the clear zone, or near the clear zone, which may need to be mitigated or protected by barrier.

All existing barrier installations are reviewed to determine if replacement and/or relocation is necessary (based on condition, height tolerance, length of need, barrier type, end treatment, etc.) or if some other mitigation measure may be more appropriate.

Refer to the Alberta Transportation “Roadside Design Guide” [5] for more details.

G.7 SUPERELEVATION

When it has been determined that horizontal curve realignment will not take place, the existing superelevation is evaluated to determine if it is appropriate for the given curve radius and design speed.

G.7.1 Consideration for Rural Highways

The superelevation rate on an existing curve is compared to what is required for the design speed and existing radius, according to new construction standards. Typically, Alberta Transportation uses a maximum superelevation rate of 0.06 m/m for all rural roads. The superelevation rate for the design speed and radius is indicated in Table B-3-5-3a.

Where the existing superelevation is less than what is recommended for new construction, for the design speed and radius, increasing the superelevation rate is to be considered. Examples for assessing superelevation rates are provided in Section G.7.1.2.

To decide whether superelevation adjustment is warranted, the following steps are taken:

1. Calculate the friction demand (f_{demand}) that will result on a vehicle travelling on the circular portion of the curve at the design speed. This value is compared to the maximum safe side friction factor (f_{max}) for the design speed. Refer to Chapter B.3.3, Maximum Safe Side Friction Factors.

2. If f_{demand} exceeds f_{max}, the superelevation rate is typically increased to e_{design} (i.e., to the recommended superelevation rate for new construction using Table B-3-5-3a). The range of acceptable superelevation rate is e_{3R} (shown in Table G-7a) up to e_{max} = 0.08 m/m (indicated in Table B-3-5-3b).

3. If f_{demand} is less than f_{max} and $e_{existing}$ is less than e_{design}, some improvement to the superelevation should be considered as follows:

   a. If f_{demand} exceeds 0.04 and is less than f_{max}, the superelevation should be set at least as high as indicated in Table G-7a and, preferably, set as high as the rate for new construction (e_{design}), using Table B-3-5-3a. Where the existing curve radius is less than the minimum indicated in Table B-3-5-3a (e_{max} = 0.06 m/m) for the design speed, an absolute maximum 0.08 m/m superelevation may be used (indicated in Table B-3-5-3b).

   b. If f_{demand} is less than or equal to 0.04, superelevation improvement is likely not required, though it may be desirable. Expenditure on superelevation improvement that only yields a more comfortable curve may be difficult to justify. When $f_{demand} \leq 0.04$, the factor of safety against side-slipping is high and is generally not a concern. The adoption of 0.04 as tolerable is supported by the policy of allowing f values up to 0.04 (approximately) on normal crown curves before applying superelevation.

Classification: Public

The following formulae are used to calculate R_{min} and f_{demand}:

$$R_{min} = \frac{V^2}{127\,(e_{max} + f_{max})}$$

where:

R_{min} = minimum radius of the circular curve for the design speed (m)
V = design speed (km/h)
e_{max} = the maximum superelevation (m/m)
Generally, 0.06 m/m for rural roads in Alberta. On 3R/4R projects, 0.08 m/m may be permitted.
f_{max} = the maximum allowable safe side friction factor for the design speed (Table B-3-3a)

$$f_{demand} = \frac{V^2}{127\,R} - e$$

where:

f_{demand} = the friction demand on the vehicle
V = vehicle speed (km/h)
R = radius of the circular portion of the curve (m)
e = existing superelevation (m/m)

Classification: Public

Table G-7a Minimum Superelevation Rate for Radii (for 3R/4R Projects)

Superelevation, e_{3R} (m/m)	Design Speed, V (see Note 1)							
	60 km/h	70 km/h	80 km/h	90 km/h	100 km/h	110 km/h	120 km/h	130 km/h
	R min (m) (f_{demand})	R min (m) (f_{demand})	R min (m) (f_{demand})	R min (m) (f_{demand})	R min (m) (f_{demand})	R min (m) (f_{demand})	R min (m) (f_{demand})	R min (m) (f_{demand})
NC	1420 (0, 0.04)	1930 (0, 0.04)	2520 (0, 0.04)	3190 (0, 0.04)	3940 (0, 0.04)	4765 (0, 0.04)	5670 (0, 0.04)	6655 (0, 0.04)
RC = 0.02	570 (0.03)	775 (0.03)	1010 (0.03)	1280 (0.03)	1575 (0.03)	1905 (0.03)	2270 (0.03)	2665 (0.03)
0.03	315 (0.06)	430 (0.06)	560 (0.06)	800 (0.05)	985 (0.05)	1305 (0.043)	1620 (0.04)	1905 (0.04)
0.04	205 (0.10)	300 (0.09)	390 (0.09)	535 (0.08)	790 (0.06)	1060 (0.05)	1260 (0.05)	1480 (0.05)
0.05	170 (0.12)	230 (0.12)	315 (0.11)	425 (0.10)	565 (0.09)	795 (0.07)	945 (0.07)	1110 (0.07)
0.06 (see Note 4)	130 (0.15)	190 (0.15)	250 (0.14)	340 (0.13)	440 (0.12)	600 (0.10)	750 (0.09)	950 (0.08)
0.07	130 (0.15)	175 (0.15)	240 (0.14)	320 (0.13)	415 (0.12)	560 (0.10)	710 (0.09)	890 (0.08)
0.08 (see Note 5)	120 (0.15)	170 (0.15)	230 (0.14)	300 (0.13)	390 (0.12)	530 (0.10)	670 (0.09)	830 (0.08)

Notes:

1. V is the design speed, usually estimated by the posted speed plus 10 km/h.
2. R_{min} is the suggested minimum radius for the indicated superelevation rate, design speed and allowable friction demand.
3. f_{demand} is the friction demand on a vehicle travelling on the circular portion of the curve for the indicated superelevation rate, design speed and radius.
4. The minimum radii in this row (e = 0.060 m/m) match the minimum radii in Table B-3-5-3a. f_{demand} equals the maximum allowable safe side friction factor (f_{max}) in Table B-3-3a.
5. The minimum radii in this row (e = 0.080 m/m) match the minimum radii in Table B-3-5-3b. f_{demand} equals the maximum allowable safe side friction factor (f_{max}) in Table B-3-3a.
6. The desirable superelevation rates are shown in the new construction standards, Table B-3-5-3a.
7. The superelevation values derived using this table, for a given design speed and radius, are the suggested minimum superelevation values acceptable for the existing curve.
8. Existing superelevation rates that are higher than the design superelevation rate for new construction are typically not altered unless greater than 0.08 m/m (the absolute maximum) or more than 0.02 m/m higher than the design rate for new construction. In these cases, the superelevation rate is typically lowered to the design rate for new construction.
9. The minimum radii for each superelevation rate and design speed have been calculated using an allowable friction demand that is higher than that used for each superelevation rate on new construction projects. The friction demand is, however, always less than the maximum allowable safe side friction factor for each design speed.

Classification: Public

G.7.1.1 Rationale for Table G-7a

Table G-7a table was developed based on the following principles:

1. On medium and larger radius curves, where the design superelevation rate for new construction is less than 0.04 m/m, the factor of safety against side-slip is high and the f_{demand} at design speed is relatively low. An existing superelevation rate that is lower than the design superelevation rate is allowed.

2. On curves with a smaller radius, where the design superelevation rate for new construction is between 0.04 m/m and 0.06 m/m, the allowable variance from the design superelevation is gradually reduced as the radius is decreased. At the minimum radius for a given design speed (for new construction) the superelevation rate suggested for 3R projects is the same as that required for new construction (i.e., 0.06 m/m). This is because f_{demand} at the minimum radius is equal to the theoretical maximum safe-side friction factor for the speed (based primarily on comfort considerations).

3. On curves with a smaller radius than the R $_{min}$ used for new construction (based on a maximum superelevation rate of 0.06 m/m) it is important to restrict the f_{demand} to f_{max} (the maximum safe side friction factor) where possible. Therefore, the superelevation rate may be increased up to 0.08 m/m for these small-radius curves, using Table B-3-5-3b (e_{max} = 0.08 m/m).

4. Because 0.08 m/m is considered a practical maximum for superelevation, the superelevation on an existing curve is not increased beyond 0.08 m/m. Because an existing curve with a radius less than R $_{min}$ (for new construction) can provide good service and have a reasonable safety record, a radius less than R $_{min}$ may be acceptable for existing paved roads. Similarly, the R $_{min}$ suggested for 3R projects should not be interpreted as an absolute minimum radius, but rather as a benchmark value. Radii below the benchmark value should be evaluated for realignment; however, realignment may not be cost effective, especially on lower volume highways. Overlay of the existing alignment (possibly with the addition of speed advisory signs) should not be ruled out. This may be the most viable alternative when all factors including construction costs, road user costs and collision costs are considered.

The following summarizes the rules used to set up the values for R $_{min}$, for each superelevation rate, for a design speed of 110 km/h. Similar rules were used for the other speeds.

e_{3R} for V = 110 km/h (design speed)

$$\mathrm{R}_{\min} = \frac{V^2}{127\ (e_{max} + f_{max})}$$

NC (normal crown) (one side of crown is -0.02m/m and one side of crown is +0.02 m/m)

	allowed until f_{demand} = 0.02 ± e	
	(f_{demand} = 0.00 for 0.02 – e)	R = 4764 ⇒ 4765
	(f_{demand} = 0.04 for 0.02 + e)	R = 4764 ⇒ 4765
e_{3R} = 0.02 m/m	(RC = reverse crown)	
	allowed until f_{demand} = 0.03	R = 1906 ⇒ 1905
e_{3R} = 0.03 m/m	allowed until f_{demand} = 0.043	R = 1305 ⇒ 1305
e_{3R} = 0.04 m/m	allowed until f_{demand} = 0.05	R = 1059 ⇒ 1060
e_{3R} = 0.05 m/m	allowed until f_{demand} = 0.07	R = 794 ⇒ 795
e_{3R} = 0.06 m/m	allowed until f_{demand} = f_{max} = 0.10	R = 596 ⇒ 600
e_{3R} = 0.07 m/m	allowed until f_{demand} = f_{max} = 0.10	R = 560 ⇒ 560
e_{3R} = 0.08 m/m	allowed until f_{demand} = f_{max} = 0.10	R = 529 ⇒ 530

Classification: Public

G.7.1.2 Examples for Assessing Superelevation Rates

Example 1
Given:
Design speed, V = 110 km/h
Existing horizontal radius, R = 750 m
Existing superelevation, $e_{existing}$ = 0.045 m/m

What is the acceptable 3R/4R range for existing superelevation?
What is the desirable rate for superelevation?
What is the recommended treatment?

Analysis:
Calculate the friction demand (f_{demand})

$$f_{demand} = \frac{110^2}{127 * 750} - 0.045 = 0.082$$

The superelevation rate for new construction (e_{design}) derived from Table B-3-5-3a (for R = 750 m and V = 110 km/h), is 0.056 m/m. As indicated in Table B-3-3a, f_{max} is 0.10.

See Point 3a in Section G.7.1. Since f_{demand} (0.082) is less than f_{max} (0.10) and $e_{existing}$ (0.045 m/m) is less than e_{design} (0.056 m/m), the existing superelevation rate should be increased to at least the value indicated in Table G-7a and, preferably, set as high as the rate for new construction (e_{design}), using Table B-3-5-3a.

From Table G-7a, e_{3R} = 0.052 m/m (rounded from 0.0523). This is the superelevation value for R = 750 m, which is interpolated between R = 795 m (e = 0.05 m/m) and R = 600 m (e = 0.06 m/m) for V = 110 km/h.

As indicated in Note 8 for Table G-7a, the highest value for existing superelevation is 0.02 m/m higher than the recommended rate for new construction. As indicated above, the recommended rate for new construction (e_{design}) is 0.056 m/m. The highest acceptable value for $e_{existing}$ is 0.056 + 0.020 = 0.076 m/m.

The acceptable 3R/4R range for $e_{existing}$ is 0.052 m/m to 0.076 m/m.

The desirable rate is the new construction rate (e_{design}) of 0.056 m/m.

Because $e_{existing}$ is less than the acceptable 3R/4R range for $e_{existing}$, the superelevation should be set at the desirable rate for new construction (e_{design}) which is 0.056 m/m.

Example 2
Given:
Design speed, V = 110 km/h
Existing horizontal radius, R = 600 m
Existing superelevation, $e_{existing}$ = 0.050 m/m

What is the acceptable 3R/4R range for existing superelevation?
What is the desirable rate for superelevation?
What is the recommended treatment?

Analysis:
Calculate the friction demand (f_{demand})

$$f_{demand} = \frac{110^2}{127 * 600} - 0.050 = 0.109$$

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The superelevation rate for new construction (e_{design}) derived from Table B-3-5-3a (for R = 600 m and V = 110 km/h), is 0.060 m/m. As indicated in Table B-3-3a, f_{max} is 0.10.

See Point 2 in Section G.7.1. Since f_{demand} (0.109) exceeds f_{max} (0.10), the existing superelevation rate should be increased to e_{design} (i.e., to the recommended superelevation rate for new construction using Table B-3-5-3a). The range of acceptable superelevation rate is e_{3R} (shown in Table G-7a) up to e_{max} = 0.08 m/m (in Table B-3-5-3b).

From Table G-7a, e_{3R} = 0.06 m/m. This is the superelevation value for R = 600 m and V = 110 km/h.

As indicated in Note 8 for Table G-7a, the highest value for existing superelevation is 0.02 m/m higher than the recommended rate for new construction. As indicated above, the recommended rate for new construction (e_{design}) is 0.060 m/m. The highest acceptable value for $e_{existing}$ is 0.060 + 0.020 = 0.080 m/m.

The acceptable 3R/4R range for $e_{existing}$ is 0.060 m/m to 0.080 m/m.

The desirable rate is the new construction rate (e_{design}) of 0.060 m/m.

Because $e_{existing}$ is less than the acceptable 3R/4R range for $e_{existing}$, the superelevation should be set at the desirable rate for new construction (e_{design}) which is 0.060 m/m.

<u>Example 3</u>

<u>Given:</u>
Design speed, V = 110 km/h
Existing horizontal radius, R = 550 m
Existing superelevation, $e_{existing}$ = 0.055 m/m

What is the acceptable 3R/4R range for existing superelevation?
What is the desirable rate for superelevation?
What is the recommended treatment?

<u>Analysis:</u>
Calculate the friction demand (f_{demand})

$$f_{demand} = \frac{110^{2}}{127 * 550} - 0.055 = 0.118$$

Table B-3-5-3a, indicates the minimum curve radius is 600 m for a design speed of 110 km/h. When it is not feasible to improve the horizontal curve radius, Table B-3-5-3b (e_{max} = 0.08 m/m) is used to obtain the superelevation rate for new construction. The superelevation rate for new construction (e_{design}) derived from Table B-3-5-3b (for R = 550 m and V = 110 km/h) is 0.079 m/m. As indicated in Table B-3-3a, f_{max} is 0.10.

See Point 2 in Section G.7.1. Since f_{demand} (0.118) exceeds f_{max} (0.10), the existing superelevation rate should be increased to e_{design} (i.e., at the recommended superelevation rate for new construction using Table B-3-5-3b). The range of acceptable superelevation rate is e 3R (shown in Table G-7a) up to e_{max} = 0.08 m/m (in Table B-3-5-3b).

From Table G-7a, e_{3R} = 0.073 m/m (rounded from 0.0733). This is the superelevation value for R = 550 m, which is interpolated between R = 560 m (e = 0.07 m/m) and R = 530 m (e = 0.08 m/m) for V = 110 km/h.

The acceptable 3R/4R range for $e_{existing}$ is 0.073 m/m to 0.079 m/m.

The desirable rate is the new construction rate (e_{design}) of 0.079 m/m.

Because $e_{existing}$ is less than the acceptable 3R/4R range for $e_{existing}$, the superelevation should be set at the preferable rate for new construction (e_{design}) which is 0.079 m/m. The minimum rate is e_{3R} (0.073 m/m).

Classification: Public

G.7.2 Consideration for Urban Roads

Refer to Chapter B.3.5.2.2., Urban Superelevation Rates, for more information.

Classification: Public

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Classification: Public