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Risk-Based Geometric Design for Road Improvements

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Risk-Based Geometric Design for Road Improvements

Summary Report of Phase 1 Study

April 2019

Risk-Based Geometric Design for Road Improvements

Summary Report of Phase 1 Study

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SUMMARY

Objectives

The *Risk-Based Geometric Design (RibGeom)* methodology for minor road improvement schemes is intended to contribute to the Strategic Goal of Transport Infrastructure Ireland (TII) for the

"maintenance and operation of safe, efficient and sustainable networks of national roads"

in accordance with the TII Statement of Strategy.

Enhancing the management of this strategic road asset is the Key Objective, and this study has sought to develop the following tools and techniques for a holistic methodology of minor improvement schemes incorporating a proactive approach:

a) Development of a Geometric Model of the Irish legacy road network so that the existing alignments can be analysed and high-risk locations for collisions identified, evaluated and prioritised for interventions to improve operational performance and road safety in a targeted and cost-effective manner;

b) An Operational Speed Model to illustrate the variable speed consistency along routes that is a contributor to collision risk;

c) A Risk Model that enables a multi-criteria assessment of collision risk based on 7 controlling factors;

d) A tool to assess and quantify the risk-reduction benefits that may be derived from potential realignments or other changes to the road geometry;

e) Inform Network Improvement Strategies through evaluation of Safety Benefits for various levels of investment across the national road network;

f) Inform a review of Design Standards for potential expansion of *DN-GEO-03030 Guidance on Minor Improvements to National Roads* to include new techniques and processes.

Geometric Risk Analysis Model

International research in a wide range of countries has advanced the development of mathematical techniques to predict the likelihood of loss-of-control type vehicle collisions linked to road geometry consistency and speed. Modelling processes have been developed to simulate the feedback loop between the road alignment and the driver along an extended section of road which enables evaluation of the risk of a vehicle being unable to safely complete the manoeuvre principally due to misinterpretation of the prevailing geometrics.

The *RibGeom* project has built upon the International State of the Art (i.e. published academic sources, guidance and initiatives by other road authorities) in developing its risk analysis methodology.

Since there are considerable variances between the quality and characteristics of road networks and driving cultures in different countries, it is necessary to develop regionally specific risk models that can be calibrated against the real performance of the road network in terms of traffic speeds and collision history.

The *RibGeom* project has considered 7 distinct factors in the context of the vehicle stability equation, namely:

- horizontal curvature;
- vertical alignment;
- forward visibility;
- road cross-fall and surface friction;
- driver workload (alertness and degree of active engagement) and,
- vehicle speed.

The *RibGeom* project has developed an integrated suite of algorithms in a single model that combines the multiple factors and provides a single risk rating result. This allows the locations of abnormal risks to be identified along a section of road so that interventions to reduce collision risk may be developed.

Risk Analysis Process

The developed risk analysis process comprises three stages:

1) Firstly, the geometric parameters of the road are derived from GPS (Global Positioning System) survey data, using a geometry definition model. This obviates the need for a topographical survey at the initial stage. The model also estimates the forward sight distance along the road.

2) Secondly, an operating speed model is used to estimate vehicle operating speeds, based on the geometric data.

3) Finally, the risk model combines the output of the first two models with various other data to produce a risk value for each road alignment element

Geometric consistency is the key characteristic of the road layout that defines collision risk. It has been found that inconsistent road geometry is a contributing factor to driver error resulting in excess vehicle speed for the road conditions, which will then raise the risk of loss-of-control. Thus, a single sharp bend on an otherwise good quality road alignment will have a higher risk rating than a similar curve on a generally bendy road. The consistency assessment results are ranked for each parameter for a section of road on a 5-point scale based on variation ranges as “Very Good”, “Good”, “Fair”, “Poor”, or “Very Poor”.

To facilitate interpretation, the risk parameters calculated by the risk model are normalised to produce “quality parameters” between 0.0 and 1.0, where 0.0 represents the lowest risk and 1.0 the highest. Typical average risk-ratings are in the range of 0.2 to 0.5, with localised spikes of up to 0.8 or 0.9 occurring at particularly poor-quality road layout locations. A risk-rating of 0.0 never occurs as there is always an inherent degree of risk due to the many complex factors involved for a vehicle travelling along the road.

Pilot Site Analysis

For the development of suitable speed and risk models for the Irish road network, a set of 30 pilot sites were selected for data collection and risk evaluation. These sites are located on national secondary roads in the west and south of Ireland of varying geometric quality and traffic flows. Speed measurements were undertaken (discreetly) at each bend and on the approach. This data was used to calibrate the Operating Speed Model. Risk ratings were developed for each pilot site and these were compared to the available collision records for validation.

Case Studies

Three case studies were undertaken to illustrate the application of the *RibGeom* methodology to a sample of 3 existing national secondary routes. These case studies provide an initial illustration of how the differing potential safety benefits may be quantified for a range of possible road improvement measures.

One case study for example, found 5 localised high-risk locations (>0.6 rating compared to 0.35 average) on the 23km long route. The cumulative length of these high-risk sections of road is 2.5km, or a little over 10% of the total length.

The case study results indicate that the best option in terms of reduced collision risk can be achieved through quite short realignments of relatively tight radius horizontal curvature, with associated relaxations from standards, to provide the best fit for consistency with the overall character of the route over the adjoining 2km long approach sections, as recommended in TII Design Standard *DN-GEO-03030 Guidance on Minor Improvements to National Roads*.

Conclusions

RibGeom provides a powerful tool to Assess Potential Realignments or other changes to the road geometry in terms of their risk rating and to quantify the risk-reduction benefits that may be derived.

At the end of Phase 1 of this project the *RibGeom* tool is well developed for the analysis of the primary risk factors associated with the horizontal road alignment. According to the EU Technology Readiness Level (TRL) scales, the tool could be considered to be at Level 6, i.e. “*Technology Demonstrated in Relevant Environment*”. However, further refinement remains for the vehicle stability factors associated with road surface friction, for which there is a separate parallel research and development programme underway by TII. The role of visibility

also requires further investigation, as does a higher level of correlation between the 7 controlling variables.

RibGeom may next be used to inform Network Improvement Strategies through evaluation of Operational and Safety Benefits for various levels of investment. Baseline studies can be undertaken to evaluate the performance of the entire legacy national road network in terms of alignment quality, speed levels and consistency, and collision risks. These exercises would be a considerable undertaking in terms of the volume of data to be processed.

Ultimately the development of *RibGeom* may inform the enhancement of TII Design Standards for potential expansion of *DN-GEO-03030 Guidance on Minor Improvements to National Roads* to include new techniques and processes.

With the *RibGeom* methodology Transport Infrastructure Ireland can adopt international best practice and latest research in embedding risk-based methodologies at the heart of its asset management practices, which could achieve very significant improvements for road operation and safety at optimised costs, and in a targeted and environmentally sustainable manner. Thus, it would enable TII to make substantial progress towards the Strategic Goal for *"the provision, maintenance and operation of safe, efficient and sustainable networks of national roads"*.

Future Developments

In addition to the applications of the *RibGeom* methodology at network level as summarised above, a scope of work for Phase 2 of the project has been developed to advance the work to date for the following aspects:

1) Regional variability of collision risk factors according to topography, traffic density and driver expectations;

2) Expanded Speed Data for broader validation of the Operational Speed Model with better statistical reliability;

3) Dynamic Speed Profile Data to improve the fit of the Speed Model to the variation of alignment quality at transitions;

4) A process for Speed Limits Review for closer fit to the true operational speeds on sub-groups of the National (and possibly Regional) Road networks;

5) Collision Risk associated with Junctions.

6) Incorporate work from parallel research strands associated with (i) collision models, (ii) stopping sight distance assessment and (iii) temporal friction modelling.

The aim of Phase 2 of the project is to raise the Technology Readiness Level of the RibGeom tool to TRL 8, i.e. System Complete and Qualified, whereby it will become a mainstream tool for decision making and asset management.

1. OBJECTIVES AND SCOPE

1.1 Context

The National Road Network in Ireland is 5,300 km in length, of which a little over 40% consists of motorway, dual carriageway and high-quality single carriageway roads that were designed and constructed to engineering standards. These roads carry the highest volumes of traffic and provide a consistent geometry in terms of alignment, width and visibility so that drivers can maintain high speeds and experience a consistent quality of service. The “driver workload” on these roads is low, such that drivers may adopt a relaxed approach to their task with little need for hard braking and gear changes. Generally, such roads have a good safety record, though on the single carriageways where traffic volumes are high there may be increased risk of head-on collisions during overtaking manoeuvres. Average traffic speeds are normally close to the 100 km/h speed limit on these roads.

The remaining 60% of the network, with a length of approximately 3,000 km, carries considerably lower volumes of traffic, and consists of “legacy” roads that have evolved over time through successive and gradual widening and improvements for pavement quality, road markings and traffic signs. These roads are of variable and often poor quality in terms of their geometry, sometimes with very severe bends that are highly inconsistent with the preceding alignment. As a result, vehicle speeds vary significantly and the “driver workload” is high with regular sharp braking and acceleration. The road width is usually quite narrow in the range of 7m to 6m or less and without hard shoulders and with narrow verges that restrict forward visibility. While a 100 km/h speed limit applies to all national roads, on the legacy roads the average speed is often 80 km/h or lower. Some routes in mountainous areas along the western seaboard have average speeds less than 60 km/h.[1]

Due to the poor and highly variable road geometry on legacy roads there are regular high risks of collisions due to loss of control by drivers who may be travelling at excessive speed for the road conditions, or who struggle to quickly adjust their speed in response to sudden changes in the road alignment. Single vehicle collisions are more frequent on these roads and this is reflected in poor safety ratings with many routes ranked as having more than twice the expected collision rate.

1.2 Challenges fo a Road Improvement Strategy

For the long length of the legacy road network it is not realistic to plan for all of it to be improved to a uniform standard. Typical improvement scheme costs are in the range of €3 million to €5 million per km for a single carriageway with a mix of on-line widening works and off-line realignments at sharp curves. On this basis a full improvement programme for the 3,000 km would cost in the region of €12 billion. At an annual investment rate of €50 million to €100 million for such roads (as has been typical over

[1] Source: National Roads 2040 Strategy Working Papers based on national traffic speeds data from mobile phone signals.

recent decades) it would only be possible to improve perhaps 30 km per annum and a full programme could never be completed. Clearly a more affordable and realistic improvement strategy is required to achieve meaningful benefits in a shorter timeframe.

About half of the legacy national road network carries low traffic volumes in the range below 5,000 vehicles per day AADT. Traffic flows on the rest of the legacy network is in the range of 5,000 to 10,000 AADT. Below 5,000 AADT it is difficult to achieve sufficient economic benefits to justify the high cost of generalised improvement to current road design standards for a consistent design speed of 100 km/h. The adoption of the narrower Type 3 Single Carriageway in the past decade and application of a lower design speed of 85 km/h has assisted somewhat in enabling improvement schemes to retain more of the old road and to fit better with the terrain while reducing costs to a degree, but the affordability challenge remains considerable.

A different perspective is needed to provide a meaningful approach to targeted road improvements that can achieve realistic benefits in a shorter timeframe.

1.3 Key Objective for an Improvement Strategy for Legacy Roads

The TII Statement of Strategy sets the Strategic Goal of the organisation as to:

> *"Secure the provision, maintenance and operation of safe, efficient and sustainable networks of national roads, light rail and metro."*

Figure 1.1 - Current TII Statement of Strategy

Figure 1.2 - Extract from the TII Statement of Strategy

In the absence of specific economic objectives for the legacy road network, the key issue must be to manage and improve operational and road safety performance levels.

On this basis there is a need to focus on the proactive identification of high collision risk locations and operational inconsistencies on the legacy road network to enable targeted improvements that will achieve maximum benefits in a cost-effective manner. The TII Design Standards already provide some guidance in this regard but there is a need to develop more advanced and practical methodologies to better achieve the best outcomes for consistent improvements on a network wide basis.

This study sought to develop a suitable methodology and associated technical tools to support a risk-based approach to the design of road improvements within the framework of the existing design standards.

1.4 Road Design Standards

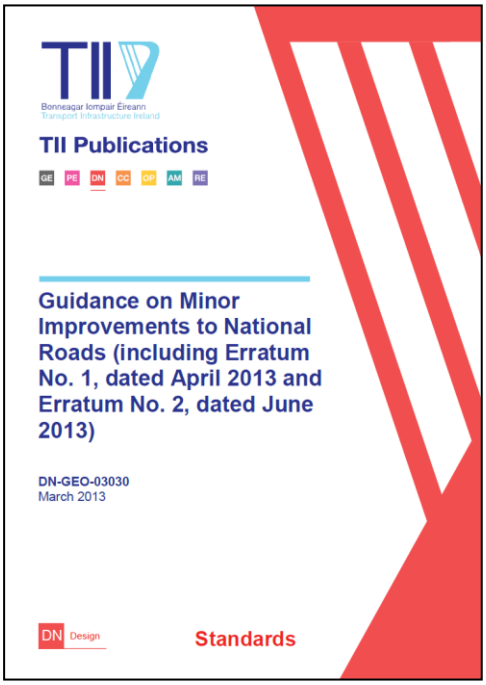
1.4.1 Applicable TII Standards

TII Publications includes two design standards that are relevant to the design of minor road safety improvements schemes:

DN-GEO-03030 Guidance on Minor Improvements to National Roads (Former TA 85)

and

DN-GEO-03031 Rural Road Link Design (Former TD 9)



The advice provided in DN-GEO-03030 is extracted below in terms of the scale and objectives for a minor scheme:

> *Minor Improvement Scheme*
>
> 1.5 A Minor Improvement Scheme is an upgrade to an existing section of sub-standard road less than 2km in length where a design element or combined set of design elements are improved. Minor Improvement Schemes vary in complexity, ranging from the removal of inappropriate adverse camber to the isolated improvement of sections of an existing road.
>
> *Road Safety Improvement Scheme*
>
> 1.6 A Road Safety Improvement Scheme is a Scheme that specifically targets sections of the network with high collision rates to improve road safety, where a design element or combined set of design elements are improved to reduce the frequency and or the severity of collisions occurring in the future.

A key parameter is associated with the consistency of the road layout:

> *Route Consistency*
>
> 1.8 Route Consistency is achieved by a route improvement appropriate to and consistent with characteristics of the existing road alignment such as the existing route geometric characteristics and traffic demand (in particular the volume of daily traffic and Heavy Commercial Vehicle (HCV) percentage).

Section 1.11 defines the design objective for a Minor Improvement Scheme as to:

> *"Achieve a localised improvement appropriate, and consistent with the characteristics of the adjacent sections of the route …."*

The above design objective can be difficult to apply effectively in practice without a technique to assess the consistency of the road layout in an objective fashion in the context where the Primary Focus is to Manage the Asset within a reasonable budget to achieve the following objective to:

Maximise Performance & Minimise Collision Risk

The Design Standard recognises this challenge and states:

> *"Many roads in Ireland are legacy roads with sub-standard design features with respect to the NRA DMRB Standards. The objective of a Minor Improvement Scheme is to upgrade some, but not all these existing deficiencies within environmental & budget constraints."*

The main objective of a scheme may be to improve some severe bends that are associated with high collision risk. The question arises as to which bends should be selected for improvement, and to what geometric standard?

Figure 1.3 - Example of a Series of <200m Radius Bends on the N14 in Donegal

1.4.2 Design Speed

The usual first step for a road alignment design is to determine the Design Speed in accordance with Section 1 of Standard DN-GEO-03031 Rural Road Link Design. This methodology may provide a seemingly higher than expected result that needs to be understood in the context of the permitted number of steps of Relaxations from Standards that may be present and the associated speed variations that are to be expected.

The actual operating speed of the traffic on the road section in question may be considerably lower than the “Design Speed”, and this is normal due to the typical variability of the road alignment to which drivers will respond by adjusting their speed appropriately. Nonetheless, wide and sudden variations in operating speed are undesirable in terms of route consistency which may have implications for road safety and accentuated collision risks.

1.4.3 Relaxations from Standards

Table 1.3 (below) and Section 3.5 of Standard DN-GEO-03031 permits the following degrees of Relaxations from Standards for Horizontal Alignment according to the road cross-section:

3 Steps for Type 2 Single Carriageway Roads

4 Steps for Type 3 Single Carriageway Roads.

TII Publications *DN-GEO-03031*
Rural Road Link Design *June 2017*

3.5 Relaxations in Horizontal Alignment

In the circumstances described in Section 1.8, Relaxations below the Desirable Minimum values may be made at the discretion of the designer. The numbers of Design Speed steps permitted below the Desirable Minimum are normally as follows:

Motorways, Dual Carriageways and Type 1 Single Carriageway Roads:	2 steps
Type 2 Single Carriageway Roads:	3 steps
Type 3 Single Carriageway Roads:	4 steps

Extract from DN-GEO-03031 for Design Speed Relaxations

Table 1.3: Design Speed Related Parameters

DESIGN SPEED (km/h)	120	100	85	70	60	V2/R
STOPPING SIGHT DISTANCE m						
Desirable Minimum Stopping Sight Distance	295	215	160	120	90	
One Step below Desirable Minimum	215	160	120	90	70	
Two Steps below Desirable Minimum	160	120	90	70	50	
Horizontal Curvature m						
Minimum R+ without elimination of Adverse Camber and Transitions	2880	2040	1440	1020	720	5
Minimum R+ with Superelevation of 2.5%	2040	1440	1020	720	510	7.07
Minimum R with Superelevation of 3.5%	1440	1020	720	510	360	10
Desirable Minimum R with Superelevation of 5%	1020	720	510	360*	255*	14.14
One Step below Desirable Min R with Superelevation of 7%	720	510	360	255*	180*	20
Two Steps below Desirable Min R with Superelevation of 7%	510	360	255	180*	127*	28.24
Three Steps below Desirable Min R with Superelevation of 7%			180	127*	90*	40
Four Steps below Desirable Min R with Superelevation of 7%			127	90*	65*	56.56
Vertical Curvature – Crest						
Desirable Minimum Crest K Value	182	100	55	30	17	
One Step below Desirable Min Crest K Value	100	55	30	17	10	
Two Steps below Desirable Min Crest K Value	55	30	17	10	6.5	
Vertical Curvature – SAG						
Desirable Minimum Sag K Value	53	37	26	20	13	
One Step below Desirable Min Sag K Value	37	26	20	13	9	
Two Steps below Desirable Min Sag K Value	26	20	13	9	6.5	
** Absolute Minimum Vertical Curve Length to be used on Dual Carriageways	240	200	-	-	-	
Overtaking Sight Distances						
Full Overtaking Sight Distance FOSD m.	N/A	580	490	410	345	
FOSD Overtaking Crest K Value	N/A	400	285	200	142	

Notes

Extract from DN-GEO-03031 for Design Speed Related Parameters

1.4.4 Typical Existing Road Alignment Compared to Design Standards

The usual conditions on the legacy routes are for an existing road cross-section of less than 7m, which is at or below the width of a Type 3 Single Carriageway. A Design Speed Assessment may provide a result of 85 km/h on a route that is very bendy. Application of the Design Standards requirements for such a situation in accordance with Table 1.3 would indicate a minimum radius of 180m at 3 steps relaxation below the Desirable Minimum Radius of 510m. This evaluation provides a lower bound for the horizontal alignment that enables an initial analysis to identify those curves that may be of concern due to being sharper than 180m radius and therefore below the minimum accommodated in the design standard.

Another situation could involve a less bendy road that achieves a Design Speed Assessment result of 100km/h but with some isolated sharp bends that are more inconsistent with the overall road alignment. In that case the lowest radius curve in accordance with the design standard at 3 steps relaxation would be 360m radius below the Desirable Minimum Radius of 720m.

1.4.5 Application of Design Standards to Minor Road Improvement Schemes

As has been outlined above, the existing road design standards accommodate a wide range of horizontal alignment quality which is aligned with the stated objectives for Minor Improvement Schemes to seek to achieve only modest alterations to the road layout as necessary for better consistency and thus to achieve improved driving performance levels.

However, there are some practical difficulties for Road Designers when seeking to apply the design standards as follows:

a) How can “Consistency” be measured so as quantify the degree of improvement that a proposed scheme may achieve?
b) Can safety benefits be measured in terms of changes to Risk of Collision?
c) Is there a method to evaluate Operating Speed to illustrate the likely impact of a proposed minor improvement scheme along a route upstream and downstream of a section proposed for improvement?
d) Can there be potential unforeseen disbenefits in terms of Risk Migration from the location of a road layout improvement to another nearby unimproved road section?

1.5 Definition of Existing Road Alignment

Detailed Mapping of the legacy routes is usually not available in advance of topographical survey which is normally undertaken once a plan has been adopted to develop a proposed minor road safety improvement scheme. It has therefore been difficult for road authorities to scientifically evaluate the road geometry and the role it may play in the cause of collision risk on a network wide basis

This study has developed a streamlined process to analyse GPS string data available from annual SCRIM surveys of the road pavements quality on the national road network. This data is recorded on the TII Geographical Information System database. From that data it has been possible using mathematical techniques to define the road alignment geometry in 3 dimensions for both the horizontal and vertical alignments. Thus, the quality of the entire road network may be analysed for the first time.

1.6 Research Basis

The relationship between road geometry and collision risk has been the subject of numerous academic studies at both University level and by the research bodies of various road authorities across numerous countries in recent decades. There are several publications that capture the results of those studies and these are described in Chapter 2 of this report.

This project has investigated international State of the Art research across a wide selection of countries to collate best practice and to identify emerging techniques and tools that can be adopted and further developed for an integrated approach to risk-based geometric design for improved management of the Irish road network.

Similar work is underway in other countries, such as the USA, by road authorities seeking to improve asset management through road design processes aimed at improved road safety on existing roads that may be of variable geometric quality. However, there are geographical and cultural differences between the road networks and driver populations in different countries, which requires localisation of various parameters in models that may be developed. For this purpose, this project involved evaluation of a sample of 30 pilot sites on National Secondary Roads in Ireland, mainly in the west and south of the country as described in Chapter 4.

1.7 Objectives for this Study

The key Objectives for this study are to develop the following tools and techniques to assist with road network management to identify road improvement requirements, and to aid the designers of minor road safety schemes as follows:

g) A Geometric Model of the Irish legacy road network so that the existing alignments can be analysed;

h) An Operational Speed Model to illustrate the variable speed consistency along target road sections;

i) A Risk Model that enables a multi-criteria assessment of collision risk;

j) A tool to Assess Potential Realignments or other changes to the road geometry and to quantify the risk-reduction benefits that may be derived;

k) Inform Network Improvement Strategies through evaluation of Safety Benefits for various levels of investment;

l) Inform Design Standards for potential expansion of *DN-GEO-03030 Guidance on Minor Improvements to National Roads* to include new techniques and processes.

For ease of reference the output of this project has been named ***RibGeom*** as an abbreviation for Risk Based Geometric Design.

1.8 Case Studies

A number of case studies are presented in Chapter 5 to illustrate the application of the *RibGeom* methodology to a sample of 3 existing national secondary routes. These case studies only provide an initial illustration of how the differing potential safety benefits may be quantified for a range of potential road improvement measures.

The results for these case studies, and of some other real projects that Roughan & O'Donovan worked on during mid-2018, indicate that the best option in some cases can be quite short realignment and of relatively tight radius horizontal curvature, with

associated relaxations from standards, to provide the best fit for consistency with the overall character of the route over the adjoining 2km long approach sections.

2. GEOMETRIC RISK ANALYSIS MODEL

This chapter describes the development of the geometric risk analysis model.

2.1 Fundamental Geometric Design Principles

Traffic collision occurrence is one of the major public concerns for government institutions and infrastructure managers around the world.

In this context, the main considerations for road designers/managers are to:

- Understand the factors that may lead to a collision;
- Define geometric consistency criteria that rank the collision risk potential of roads;
- Predict collision occurrences on different road networks;
- Estimate optimal risk mitigation measures.

Road networks are complex environments with many variables. This tends to make analysis more difficult. (Fernandes and Neves 2013) observed that modelling of collisions is often based on uniform road sections. Hence, many studies have examined the relationship between road geometric design and crash occurrence and road design consistency as a major surrogate measure of the safety level (Camacho-Torregrosa, et al. 2013), noting the following:

Design Consistency

Once the most accurate definitions of design consistency, extracted from (Wooldridge, et al. 2003), is that it is: "*The conformance of a highway's geometric and operational features with driver expectancy*", where driver's expectancy is considered as readiness to respond to situations, events and information in predictable and successful ways.

Design Inconsistency

The most appropriate definition is defined by (Russo, Mauro and Dell'Acqua 2012), and it is: *"Roadway features which can surprise drivers by violating their expectancies and increasing the chance of delayed response times, speed errors, and unsafe driving manoeuvres that may lead to higher collision risk"*.

Therefore, the main goal of providing design consistency is to increase roadway consistency and ergo safety, through improvement of comprehension and expectancy of the roads by drivers. Therefore, the principle objective of this project was to develop a risk assessment tool to:

- Facilitate Risk Based Asset Management – i.e. multivariate optimisation with respect to Performance, Cost & Risk;
- Identify the most critical locations of the legacy road network based on risk and consistency assessment;
- Examine the main influencers of this risk;

- To use the tool as a mechanism to assess and prioritise potential realignments;
- To inform network improvement strategies, and finally,
- To inform design standards.

2.2 Literature Review

An extensive literature review has been carried out to assess the relevant research which has been conducted internationally on the topic of collision risk with a sample shown below.

Figure 2.1 - Sample of International Publications

2.2.1 Road managements approach

Management using Collision Analysis

Historically the methodology used to support road management for safety is analysis of collision data as a method of highlighting potential locations for network improvements. Collision data is normally recorded by collision type, outcome and location. It is usual to select cluster sites for further analysis to support mitigation policy formation.

(Mannering and Bhat 2014) state that collision data analysis to support the formation of mitigation policies may have several shortcomings, for example, a significant one is the quality of data gathered by the relevant authorities, as (Imprialou and Quddus 2017) or (Schlögl and Stütz 2017) pointed out. They cite as an example (Yamamoto, Hashiji and Shankar 2008), who established that less severe crashes are not likely to be reported to police and thus tend not to appear in databases. Another limitation was the use of rudimentary statistical analyses in summarizing and assimilating patterns

from the database. (Park and Lord 2007) noted a tendency to use univariate counting models for only one single model response variable, while (Fernandes and Neves 2013) observed that the modelling of collisions is often based on uniform road sections; however roads networks are complex and changing environments where many variables are involved.

The studies highlight the need for quality data that is capable of being thoroughly and robustly analysed, as well as the complexity of interactions between drivers and the all of the characteristics of roads.

Management using input procedures

Alternative methods to cluster reviews have also been used in safety focused operational road management, and generally comprise analysis of the contributory effects of the road infrastructure.

(Karlaftis and Golias 2002) noted that the literature forming the State of the Art (SoA) primarily concentrated on identifying the factors affecting collision rates, rather than on prediction. They set out to (i) developed a methodology to quantitatively assess effects of a few highway characteristics on collision rates and (ii) to develop a model capable of quick prediction of rates for a specific route section. The methodology used was presented as a decision tree format and was successful demonstrated.

The essential consideration of geometric design, as stated by (Garber and Hoel 2009), is to provide smooth and safe road alignments to satisfy the requirements of both the drivers and the vehicles that use the road. Road alignment is therefore designed to ensure that, inter alia, curvature, visibility, and super elevation provided is consistent with the anticipated vehicle speeds on the road.

2.2.2 Geometric Design Consistency

Numerous studies have examined the relationship between road geometric design and crash occurrence, and road design consistency (Lamm, Choueiri, et al. 1988) is major surrogate measure of the safety level of the two-lane rural road segment (Camacho-Torregrosa, et al. 2013).

The main goal of design consistency is to increase roadway safety through improvement of comprehension and expectancy of the roads by drivers. Thus, one of the major aspects that should be considered is the design consistency of the road, because it should be considered that drivers expect to drive on rural roadways with relatively little workload, significantly lower than on urban roads, owing to the sparse interaction with other vehicles.

However, despite not having significant interplay with other users, drivers must face the other elements that exist on the road, i.e. environmental and geometric features. Thus, because both environment and geometry are “static” characteristics of the road, they can be analysed under a risk-based approach to locate those critical points which could heighten the risk of incidents.

Two of the most important reports on design consistency are (Wooldridge, et al. 2003) and (Austroads 2015), which aim to analyse the roadway elements and to find design features and collision relationships, with the aim of proposing design guidance improvement.

One of their major conclusions is that risk-based design consistency is not only related to the alignment of the road, indeed a comprehensive list of influential variables must include:

1) **Geometric features**
   a) Horizontal alignment
      i) Curve
      ii) Straight
      iii) Transition
   b) Vertical alignment
      i) Vertical Curve
   c) Sight distance
      i) Stop sight distance
      ii) Decision sight distance
2) **Roadway features**
   a) Cross section
      i) Lane
      ii) Shoulder
   b) Surface characteristic
      i) Skid resistance
      ii) Curve super-elevation
   c) Roadway Sides
      i) Signals
      ii) Clear zones
      iii) Roadway hazard
3) **Roadway items**
   a) Intersections/Driveways
      i) Type
      ii) Location
   b) Passing lanes
      i) Geometry
   c) Rail grade crossing
      i) Type
      ii) Environment
   d) Structures (Bridges and Tunnels)
      i) Section
4) **Driver**
   a) Mental workload
      i) State (overload/distracted)
      ii) Decisions
   b) Driver state
      i) Consumption (drug or alcohol)

Operating Speed

Operating speed is defined as speed at which drivers travel on a dry roadway in free flow conditions during daylight hours and it is calculated using a specific percentile of speed distribution, typically the 85th ,V_{85} (Russo, Mauro and Dell'Acqua 2012).

Different studies, (Camacho-Torregrosa, et al. 2013), (Hassan 2004), (de Almeida 2016), (Gibreel, Easa and Hassan, et al. 1999) and (Russo, Mauro and Dell'Acqua 2012) have observed that operating speed variation, from Design Speed or between elements, maintains a strong correlation with collision rate.

Design Speed, V_d, is the speed for which the roadway alignment is designed to ensure the roadway standards of curvature, visibility, sight distance, etc. and which shall be consistent with the anticipated vehicle speed on the road. (TII Publications 2017).

There is a concern in the literature around V_{85} prediction reliability (Hassan 2004), as there exists a large number of models which are used to estimate the operating speed. (Pérez Zuriaga, et al. 2010) indicate a lack of uniformity, whilst concern also exists about computed significance levels (Fitzpatrick, Elefteriadou, et al. 2000).

One of the main issues which leads to inconsistency prediction, as (Hassan 2004) shows, is that most of the models only consider alignment characteristics. (Andueza 2000) considers that roadway characteristics mainly influence driver speed decisions, but other factor should also be estimated, i.e. curve, tangent, lane and shoulder width, as well as posted speed and access density (Fitzpatrick, Miaou, et al. 2005).

(Pérez Zuriaga, et al. 2010) concluded the operating speed model should use radius and Curvature Change Rate (CCR) as explanatory variables. Different model assessments have been performed and conclude that the most accurate speed prediction models are those which consider horizontal and vertical alignment (Misaghi and Hassan 2003) and (Gibreel, Easa and El-Dimeery 2001).

Curvature Change Rate, CCR

Since a lack of data or misinterpretation of available data often occurs, operating speed regression models are often difficult to define. (Lamm, Beck, et al. 2007) define the parameter Curvature Change Rate, CCR, which allows estimation of the Operation Speed, V_{85}, according to a relation equation as described below.

The formula for determining the Curvature Change Rate of the Single Curve with transition curves is given by the following equation:

$$CCR_s = \frac{\left(\frac{L_{Cl1}}{2R} + \frac{L_{Cr}}{R} + \frac{L_{Cl2}}{2R}\right)}{L} \cdot \frac{200}{\pi} \cdot 10^3$$

where:

CCR_S = curvature change rate of the single circular curve with transition curves [gon/km];
$L = L_{Cl1} + L_{Cr} + L_{Cl2}$ = overall length of unidirectional curved section [m];
L_{Cr} = length of circular curve [m];
R = radius of circular curve [m];
L_{Cl1}, L_{Cl2} = lengths of clothoids (preceding and following the circular curve) [m].

This evaluation is for a curve with transition curves, but other curve alignments can be calculated (see (Lamm, Beck, et al. 2007), Section 1.1). Moreover, for straight alignments, CCR=0.

It is also possible to estimate an average curvature change rate of the single curve for an observed roadway section. The method is based on a length-related calculation of the average of the CCR_{Si}:

$$\emptyset CCR_s = \frac{\sum_{i=1}^{i=n}(CCR_{si} \cdot L_i)}{\sum_{i=1}^{i=n}(L_i)}$$

$\emptyset CCR_S$ = average curvature change rate of the single curve for the observed roadway section without regarding tangents [gon/km];
CCR_{Si} = curvature change rate of the single curve or unidirectional curved site ‘i’ [gon/km];
L_i = length of curve or unidirectional curved site ‘i’ [m].

Additionally, taking into account the workload, a parameter that could be correlated with the risk of a section, is the average curvature change rate:

$$\overline{CCR} = \frac{\sum_{i=1}^{i=n}(CCR_i \cdot L_i)}{L}$$

$\overline{CCR}$ = average curvature change rate of the observed roadway section, regarding the tangents [gon/km];
CCR_i = curvature change rate of alignment element “I” along the roadway [gon/km];
L_i = length of alignment element ‘i’ (either curved or straight) [m];
L = length of the studied segment [m].

Moreover, based on additional research, equations have been developed which can relate V_{85} and V_d with the CCR (Lamm, Beck, et al. 2007) or radius (Anderson, et al. 1999).

Curve Operation Speed (V_{85}) and Curvature Change Rate (CCR) relationship

1. V_{85} for longitudinal grade G≤6% and 0<CCR_s≤1600 gon/km

$$V_{85} = 105.31 + 2 \cdot 10^{-5} \cdot {CCR_s}^2 - 0.071\ CCR_s$$

2. V_{85} for longitudinal grade G>6% and 0<CCR_s≤1600 gon/km

$$V_{85} = 86 - 3.24 \cdot 10^{-9} \cdot {CCR_s}^3 + 1.61 \cdot 10^{-5} \cdot {CCR_s}^2 - 4.26 \cdot 10^{-2} \cdot\ CCR_s$$

Design Speed (V_d) and Curvature Change Rate (CCR) relation

As mentioned in (Lamm, Beck, et al. 2007), the design speed is often not known for existing alignments. Moreover, many studies conducted at the Institute for Highway and Railway Engineering of the University of Karlsruhe, Germany, revealed that existing (old) alignments were normally not constructed for an exactly defined design speed. Therefore, (Lamm, Beck, et al. 2007) presented a procedure for determining design speed for existing alignments, based on $\emptyset CCR_S$. First, they considered the estimated average 85th-percentile speed ($\emptyset V_{85}$) of the observed existing alignment as the basis for the selection of a meaningful design speed, V_d, of a roadway segment. Thus, the Design Speed estimations result in:

$$V_d \approx \emptyset V_{85} = 105.31 + 2 \cdot 10^{-5} \cdot {\emptyset CCR_s}^2 - 0.071\ \emptyset CCR_s$$

$$V_d \approx \emptyset V_{85} = 86 - 3.24 \cdot 10^{-9} \cdot {\emptyset CCR_s}^3 + 1.61 \cdot 10^{-5} \cdot {\emptyset CCR_s}^2 - 4.26 \cdot 10^{-2} \cdot\ \emptyset CCR_s$$

Safety Criteria

Safety Criterion I

This consistency indicator contemplates the variation between design speed and operating speed. Based on existing literature Criterion I is defined as:

Table 2.1 - Geometric consistency Criterion I (Lamm R. B., 2007), (Hassan, 2004), (Lamm, 1988)

Evaluation	Classification Criterion I	
	Speed difference (Km/h)	**CCRs difference (gon/km)**
Good	$\lvert V_{85} - V_d \rvert \leq 10$	$\lvert CCR_{si} - \emptyset CCR_s \rvert \leq 180$
Fair	$10 < \lvert V_{85} - V_d \rvert \leq 20$	$180 < \lvert CCR_{si} - \emptyset CCR_s \rvert \leq 360$
Poor	$\lvert V_{85} - V_d \rvert > 20$	$\lvert CCR_{si} - \emptyset CCR_s \rvert > 360$

whereby:

- Good implies geometric consistency, so no further correction is required;
- Fair implies minor geometric inconsistencies, so no realignment corrections are required but consideration of super-elevation rate, sight distance or sign corrections may be desirable;
- Poor implies that there are major geometric inconsistencies, so realignment corrections are required.

Safety Criterion II

In this criterion the consistency between successive elements is analysed, because a significant change in operating speed may produce rear, off-road, head-on and sideswipe accidents. Hence, the variation of operating speed and curvature change rate between successive elements is analysed.

Table 2.2 - Geometric consistency criterion II (Lamm R. B., 2007), (Hassan, 2004), (Lamm, 1988)

Evaluation	Classification Criterion II	
	Speed difference (Km/h)	**CCRs difference (gon/km)**
Good	$\left\|V_{85_i} - V_{85_{i+1}}\right\| \leq 10$	$\left\|CCR_{s_i} - CCR_{s_{i+1}}\right\| \leq 180$
Fair	$10 < \left\|V_{85_i} - V_{85_{i+1}}\right\| \leq 20$	$180 < \left\|CCR_{s_i} - CCR_{s_{i+1}}\right\| \leq 360$
Poor	$\left\|V_{85_i} - V_{85_{i+1}}\right\| > 20$	$\left\|CCR_{s_i} - CCR_{s_{i+1}}\right\| > 360$

Evaluation Procedure

Considering the presented models and equations to analyse the safety consistency, procedures to evaluate the roadway segment can be detailed. Based on (Lamm, Beck, et al. 2007) these analysis procedures are depicted in the flow processes of Figure 2.2 and Figure 2.3.

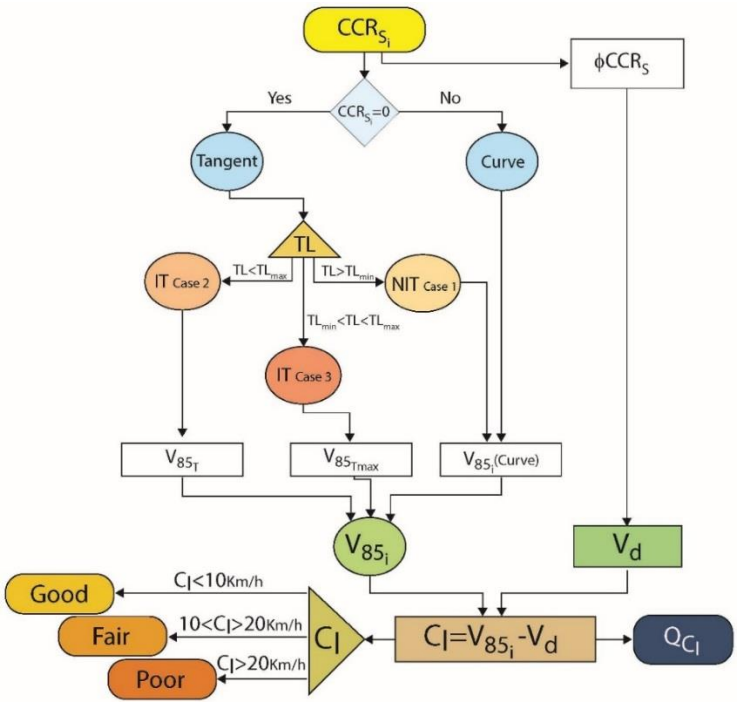


Figure 2.2 - Flow process Criterion I (based on (Lamm, Beck, et al. 2007))

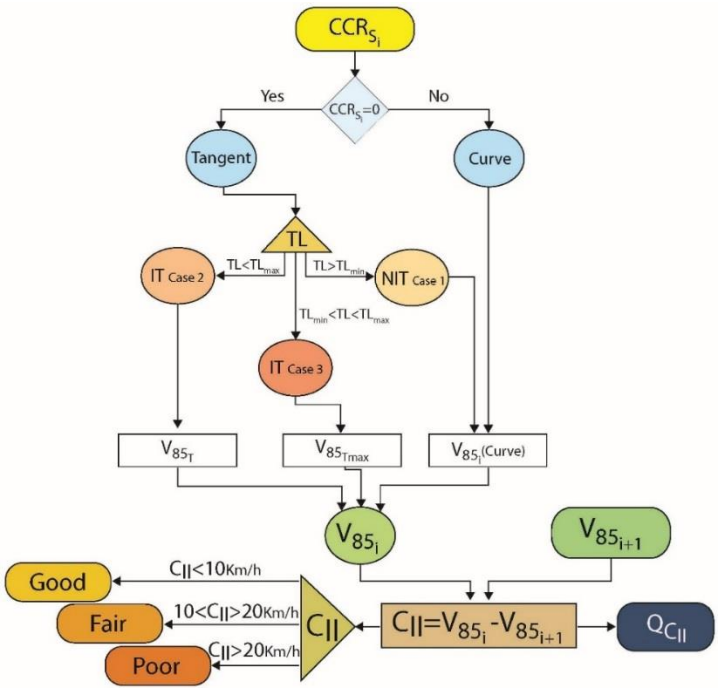


Figure 2.3 - Flow process Criterion II (based on (Lamm, Beck, et al. 2007))

Vehicle Stability

Another key parameter used in the evaluation of geometric design consistency is the analysis of vehicle stability, which is an important issue to ensure adequate skid resistance along the road. (Sullivan 2003) developed a risk-based methodology which

adopted a Monte Carlo simulation method in conjunction with International Friction Index (IFI) theory. The methodology was applied to two different sites to determine the probability that a vehicle would be able to stop in a specified stopping distance for a given speed limit for two different surfacing types. The study demonstrated that reliance on a single skid resistance measurement is often erroneous and that the proposed methodology has the potential to be used to rank road sections based on the probability that vehicles could not negotiate curves.

(Vargas Tejeda and Echaveguren 2011) subsequently developed a procedure to estimate skid resistance thresholds based on reliability theory using a conceptual model, to consider the uncertainty associated with friction demand (driver behaviour), as well as supply (resultant from seasonal variation or pavement surfacing). Two thresholds were defined in terms of skid resistance: 1) a study threshold, which corresponded to the minimum skid resistance value under which a study of collisions was required for that site, 2) an intervention threshold, which corresponded to the minimum skid resistance value under which the risk of skidding collisions increased. The procedure was demonstrated for a Chilean road network, for which sites were defined as a finite length section of road that had operational and geometric characteristics that differed from adjacent sites.

For horizontal curves, (Echaveguren, Bustos and de Solminihac 2005) proposed a probabilistic methodology to evaluate the margin of safety, which considered uncertainty associated with skid resistance measurements. In this case, friction demand was defined as the friction required by a vehicle to maintain its trajectory in an existing horizontal curve, whilst friction supply was dependant on the tire-pavement interaction and vehicle speed. The skid resistance and texture of the road surface were defined as random variables, and Monte Carlo simulation was employed to propagate the uncertainties within the analysis.

On TII's legacy network surface water drainage is generally "over the edge". There is no pipework or engineered system to ensure water flowing off the surface is transported away quickly and efficiently; rather it is expected to dissipate through soakage into the adjoining verge. This may at times of insufficient soakage lead to aquaplaning which occurs when the vehicle's tyres are separated (partially or fully) from the road surface by a film of water and which results in loss of control of the vehicle. (Horne 1968) defined two main types of aquaplaning as 'Viscous' and 'Dynamic'.

Viscous aquaplaning can occur at low speeds where the texture of the road surface is low. On the other hand, dynamic aquaplaning is the partial or full separation of tyre and pavement which can occur when the thickness of the water film on the pavement surface is such that, at a given speed, the combination of tyre tread and pavement macrotexture is incapable of discharging the bulk water from the contact patch. It may also occur if the tyre is free rolling or locked.

No specific guidance is currently provided to define at what point surface drainage problems exist. TII has decided to adopt the principles set out by (Gallaway, et al. 1979) with maximum permissible water depths established at the limit at which surface

drainage becomes a problem. The highway designer is provided with a clear set of parameters to meet using a clear step by step procedure. Use of the Gallaway formula also puts in place a methodology for highway designers to assess surface drainage issues more accurately. Although water film depths can be easily determined in locations of consistent surface geometry; the process is more complex at super-elevation rollovers. Aquaplaning potential is assessed via a two-part process:

1. Determine water film depth (particularly in the anticipated wheel paths) for a flow path across the pavement;
2. Based on operating speed of the road section, check estimated water film depth against acceptable depths limits.

Safety Criteria

Geometric design should satisfy the ratio between tyre and roadway surface, especially at curved sites. When insufficient side friction is provided at a horizontal curve, vehicles may skid out, rollover, or be involved in head-on collisions. So sliding and overturning failures may take place. Thus, the critical sliding condition is given by:

$$v^2 = rg(p \pm f_R)$$

where:

v Vehicle speed;
r Curve radius;
g Gravity acceleration;
f_R Friction coefficient;
p Super-elevation;
$\pm$ Outward or inward direction of the slide.

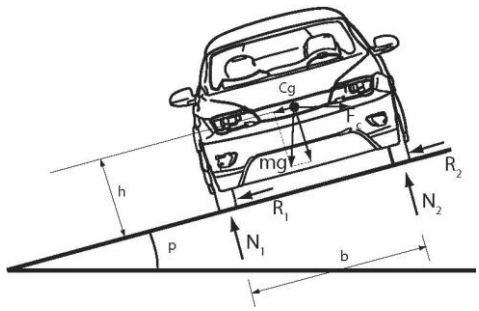


Figure 2.4 - Forces Acting on a Vehicle in a Curve

The overturning critical condition is given by:

$$\frac{b}{2}(N_2 - N_1) = \pm h(R_1 + R_2) = \pm f_R h(N_1 + N_2)$$

where

b Width of the vehicle;
h Height of the vehicle to the gravity centre;
N_1, N_2 Normal forces to the wheels;
R_1, R_2 Friction forces;
p Super-elevation.

Four types of failure mode are presented:

1. *Outward slide*:
When the vehicle slides in the outward direction of the curve due to insufficient friction or slope, a small curve radius or a high vehicle speed. This failure occurs when:

$$v^2 > rg(p + f_R)$$

2. *Inward slide:*
   When the vehicle slides in the inward direction of the curve due to insufficient friction, a high slope or a low vehicle speed. This failure occurs when:
   $$v^2 < rg(p - f_R)$$

3. *Outward overturning:*
   When the vehicle overturns in the outward direction of the curve due to insufficient width of the track or a high location of the gravity centre. This failure occurs when $N_1 = 0$, that is, when:
   $$f_R > \frac{b}{2h}$$

4. *Inward overturning*:
   When the vehicle overturns in the inward direction of the curve due to high width of the track or to a high location of the gravity centre. This failure occurs when $N_2 = 0$, that is, when:
   $$f_R > \frac{b}{2h}$$

Consequently, (Lamm, Beck, et al. 2007), (Hassan, Sayed and Tabernero 2001), (Ng and Sayed 2004) and (Watters and O'Mahony 2007) present a design consistency criterion, Criterion III, which compares side friction provided by infrastructure (f_{RA}) for curve design with side friction demanded (f_{RD}) for cars riding through the curve at the 85th-percentile speed level.

Table 2.3 - Geometric consistency Criterion III (Lamm R. B., 2007), (Hassan, 2004), (Lamm, 1988)

Evaluation	Classification Criterion III	
	Side friction difference	**CCRs difference (gon/km)**
Good	$f_{R_A} - f_{R_D} \leq +0.01$	$CCR_{s_i} \leq 180$
Fair	$-0.04 \leq f_{R_A} - f_{R_D} < +0.01$	$180 < CCR_{s_i} \leq 360$
Poor	$f_{R_A} - f_{R_D} < -0.04$	$CCR_{s_i} > 360$

where:

f_{R_D} Side friction demanded by vehicles

$$f_{R_D} = \frac{V_{85}^2}{127 \cdot r} - p$$

f_{R_A} Side friction assumed by road

$$f_{R_a} = n \cdot 0.925 \cdot f_t$$

f_t Side friction factor $0.59 - 4.85 \cdot 10^{-3} \cdot V_d + 1.51 \cdot 10^{-2} \cdot V_d$

n Utilization ratio (0.40, 0.45 for new design with hilly or flat topography, or 0.6 for existing old alignment)

Evaluation process

The procedure to define the Criterion III is presented in Figure 2.5.

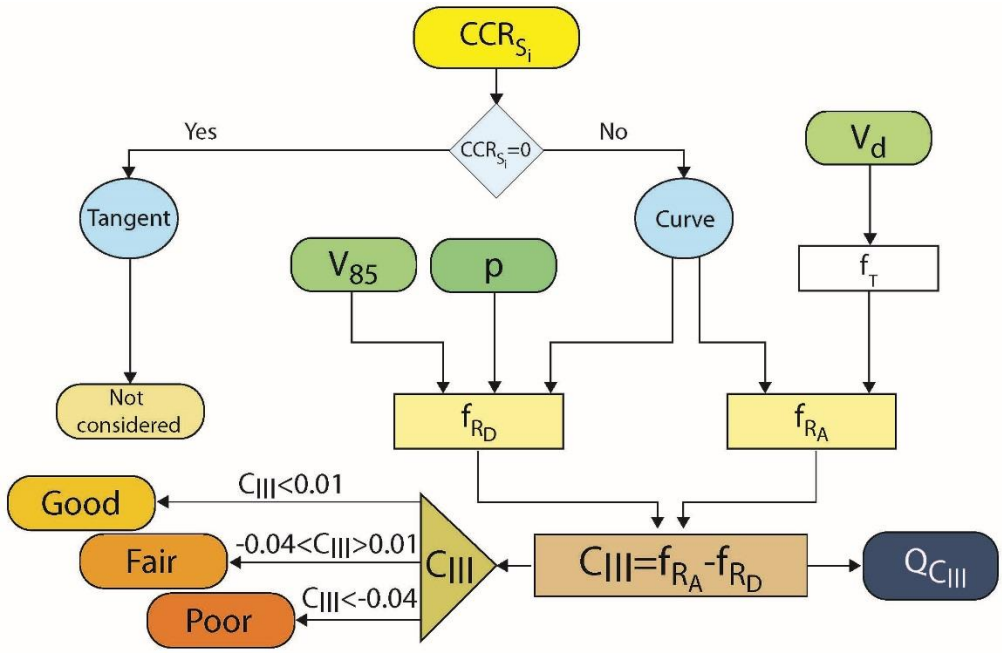


Figure 2.5 - Flow process Criterion II (based on (Lamm R. B., 2007))

Alignment Indices

Alignment indices are quantitative measures of the general character of a roadway segment's alignment that appear to have several conceptual advantages for use in design consistency evaluations (Fitzpatrick, Woolridge, et al. 2000) as well as a direct correlation with accident rates. Various alignment indices proposed in the Literature were examined for their applicability for use in design consistency by (Fitzpatrick, Woolridge, et al. 2000). Those recommended indices are outlined in the following sections.

Table 2.4 - Alignment Indices (Fitzpatrick, Woolridge, et al. 2000)

Horizontal Alinement Indices	Vertical Alinement Indices
• Curvature Change Rate - CCR (deg/km) $\frac{\sum \Delta_i}{\sum L_i}$ where: Δ = deflection angle (deg); L = length of section (km)	• Vertical CCR - V CCR (deg/km) $\frac{\sum A_i}{\sum L_i}$ where: A = absolute difference in grades (deg); L = length of section (km)
• Degree of Curvature - DC (deg/km) $\frac{\sum (DC)_i}{\sum L_i}$ where: DC = degree of curvature (deg); L = length of section (km)	• Average Rate of Vertical Curvature - V AVG K (km/percent) $\frac{\sum \frac{L}{\lvert A \rvert}}{n}$ where: L = length of section (km); A = algebraic difference in grades (%); n = number of vertical curves
• Curve Length: Roadway Length - CL:RL $\frac{\sum (CL)_i}{\sum L_i}$ where: CL = curve length (m); L = length of section (m)	• Average Gradient - V AVG G (m/km) $\frac{\sum \lvert \Delta E_i \rvert}{\sum L_i}$ where: ΔE = change in elevation between VPI_{i-1} and VPI_i (m); L = length of section (km)
• Average Radius - AVG R (m) $\frac{\sum R_i}{n}$ where: R = radius of curve (m); n = number of curves within section	**Composite Alinement Indices** • Combination CCR - COMBO (deg/km) $\frac{\sum \Delta_i}{\sum L_i} + \frac{\sum A_i}{\sum L_i}$ where: Δ = deflection angle (deg); A = absolute difference in grades (deg); L = length of section (km)
• Average Tangent - AVG T (m) $\frac{\sum (TL)_i}{n}$ where: TL = tangent length (m); n = number of tangents within section	

Horizontal Alignment indices

The horizontal alignment indices recommended are indicators of the amount of bendiness of the road, because:

- Sharp curves may produce drastic speed reduction and difficult manoeuvrability;
- Long tangents are not conducive to controlling maximum speeds and may increase drivers' distraction and fatigue.

Both of these cases can lead to accident rate increases.

Radii measures of alignment indices are based upon the radii for the curves on the road and provide an indication of the sharpness of the curves along the roadway, i.e.:

i. *Average Radius (R_{avg})* (Hassan, Sayed and Tabernero 2001)

$$R_{avg} = \frac{\sum R_i}{n}$$

where:

R_i Radii of individual curve;

n Number of curves within section.

ii. *Ratio of Radius and Average Radius (CRR_i)* (Hassan, Sayed and Tabernero 2001)

$$CRR_i = \frac{R_i}{R_{avg}}$$

Curvature dispersion and drivers' expectancy violation lead to potential inconsistency and collision rate increase. These indices are considered to have better statistical significance with accident frequency than tangent length rates, so the tangent indices are neglected hereafter.

Vertical Alignment indices

Vertical alignment indices are useful to determine the hilliness of the road and in locating vertical curves with significant variation which can produce design inconsistencies. The amount of hilliness on an alignment can affect the speed, available overtaking distance and consequently, the accident rate. (Fitzpatrick, Woolridge, et al. 2000).

1. *Average* Vertical Curvature (VR_{avg})

$$VR_{avg} = \frac{\sum k_i}{n}; \qquad k_i = \frac{L_i}{|A|}$$

where:

L_i Length of vertical curve;
A Algebraic difference in grades (%);
n Number of vertical curves.

2. *Ratio* Vertical Curvature *and Average* Vertical Curvature (VRR_i)

$$VRR_i = \frac{\frac{L_i}{|A|}}{\frac{\sum \frac{L_i}{|A|}}{n}}$$

If this ratio shows a significant variation it may indicate a high-grade variation that can generate uncomfortable driving or lack of appropriate sight distance, which can lead to geometric inconsistencies and increased accident rates.

Stopping Sight Distance

Stopping sight distance (SSD) is commonly defined in standards as the measured distance from a driver's eye height to an object height above the road surface. It is the distance needed by a vehicle to make a complete stop before it reaches the viewed object. SSD has two components emanating from (i) perception / reaction time and (ii) braking distance. In road design the latter is normally simplified by assuming that deceleration is caused solely by surface friction. For safe and efficient operation of a vehicle, a driver needs to have sufficient sight distance (SD) length to avoid colliding with unexpected objects. (Bella 2014) confirms the need to look at an alternative

method of calculating in situ stopping site distance to that of a traditional 2D approach. However, using LiDAR datasets (Tarel, et al. 2012) to develop a ray-tracing algorithm using a triangulated 3D surface, or using the Available Sight Distance technique for visibility analysis proposed by (Hassan, Easa and Abd El Halim 1996), requires a high degree of computation.

Driver Workload

Driving is a task which requires a mental workload by drivers, and a direct relation has been observed between locations with high workload or large positive change in workload and high accident rate, see (M. D. Wooldridge 1994). Driver workload must therefore be included in any risk model. For the sake of clarity the following aspects are defined, which have been extracted from (de Waard 1996) and (Bongiorno, et al. 2017).

Task demand

This is the goal that has to be attained by means of task performance. It is external to and independent of the individual.

Load or Workload

The effect the demand has on the operator in terms of stages that are used in information processing and their energetics, i.e. amount of information processing capacity that is used for task performance. It differs between individuals, depending on:

- *Complexity demand (external):* the amount of processing that is required to perform a task.
- *Difficulty of a task (internal):* the amount of resources that are required by the individual for task performance, dependent upon context, stage, capacity and strategy or policy of allocation of resources.

As well as workload one must consider the ratio between used and available resources for a task performance developed by an individual, i.e.:

Effort

Effort is a voluntary mobilization process of resources:

- *State-related effort* is exerted to maintain an optimal state of task performance;
- *Task-related effort* is exerted in the case of controlled information processing.

Although the primary task of the driver is defined as *"safe control of the vehicle within the traffic environment"*, driving is described as a complex task encompassing processes which at a minimum of three hierarchical levels (de Waard 1996):

1. *Strategic level*: strategic decisions are made, such as the choice of means of transport, setting a route goal, and route-choice while driving. Controlled processing.

2. *Manoeuvring level:* reaction to local situations including reaction to the behaviour of other traffic participants. Controlled processing.
3. *Control level*: the basic vehicle control processes occur, such as lateral-position control. Automatic processing.

The *Task Difficulty* depends upon task complexity, the operator's capabilities, his/her state and the applied strategy. A relation between task demand and task performance is employed to determine the driver's effort, as illustrated in Figure 2.6 (de Waard 1996).

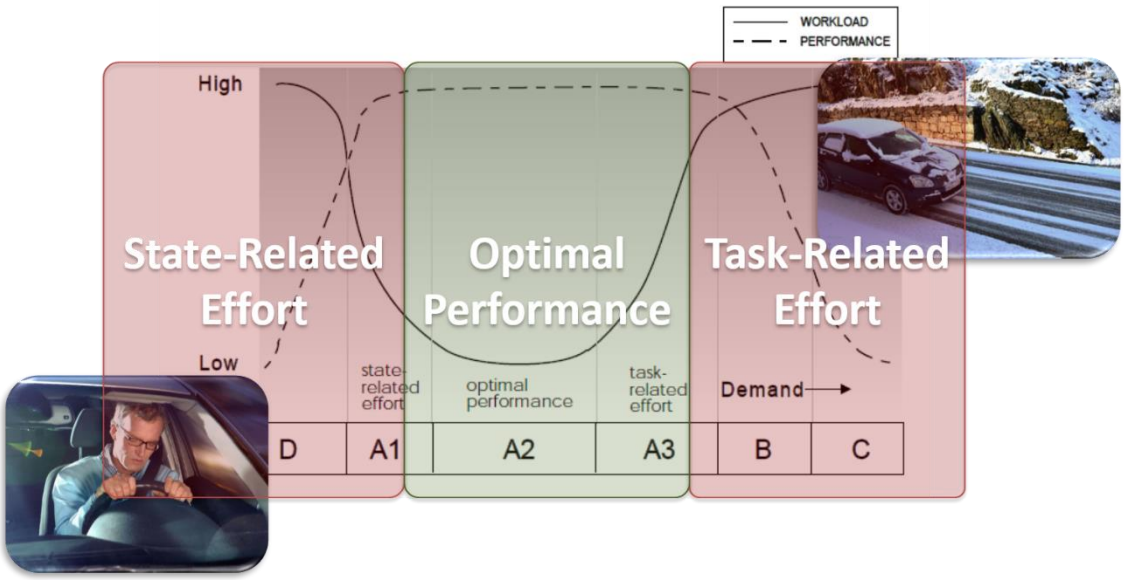


Figure 2.6 - Driver Effort

As (Gibreel, Easa and Hassan, et al. 1999) or (Krammes and Glascock 1992) defined, there is a subjective rating scale, suggested by (Messer, Mounce and Brackett 1981) and (C. Messer 1980), which ranges from 0 (good consistency) to 6 (big design inconsistency), and estimates the driver workload of each specific feature of the roadway with the following equation:

$$\mathrm{WL}_n = \mathrm{U} \cdot \mathrm{E} \cdot \mathrm{S} \cdot \mathrm{R}_f + \mathrm{C} \cdot \mathrm{WL}_{n-1}$$

where:

WL_n	Workload of specific feature;
U	Driver familiarity factor;
E	Feature expectancy factor;
S	Sight distance factor;
R_f	Basic workload potential rating;
C	Carryover factor, and
WL_{n-1}	Workload of the previous feature.

For geometric inconsistency arising from driver workload (WL≥6), the level of consistency can be aided by increasing spacing between geometric features and increasing sight distance to the feature (Gibreel, Easa and Hassan, et al. 1999).

2.2.3 Alignment Data

While high-quality GPS data is available for the Irish National Route network, converting these x, y, z coordinates to road geometric parameters (such as horizontal and vertical curve radius) is not a straightforward process. Various approaches have been described in the literature, with no clear standard method. Broadly speaking, the methods described fall into two distinct approaches: those that attempt to mathematically fit a circle to three or more consecutive GPS points, and those that examine the changes in bearing/azimuth between successive points.

A simple approach (Ambros and Valentova 2016) is to consider each point along with the point on either side of it – by considering an arc through these three points, a radius can be determined. (Hans, Souleyrette and Bogenreif 2012) also used similar methods but concluded that the results were only suitable for limited applications, unless improved by input from a human operator. (Di Mascio, et al. 2012) propose a method using a higher number of points at a time, while (Ai and Tsai 2015) developed a GIS[2]-based method which varied the number of points in different circumstances.

Most of these methods either produce somewhat “noisy” results (where small errors in the GPS data cause significant variations in the output), or they are semi-automatic, requiring human input to determine the start and end points of alignment elements.

(Li, et al. 2012) describe an alternative approach that involves examining the azimuth (bearing) of the points to determine the alignment parameters. This method was found to work quite well but had a tendency to miss reverse curves in certain circumstances. (Karamanou, et al. 2010) also describe an azimuth-based method but concluded that it needed human input to correctly identify the start and end of alignment elements.

2.3 Risk Parameters

The literature reports two principal trends which have been developed in geometric design consistency assessment, namely:

- Discrete analysis, whereby all the alignment elements of the road are analysed, e.g. (Lamm, Beck, et al. 2007), and
- Global assessment, which determines the global consistency of a segment as a unique section, e.g. (Camacho-Torregrosa, et al. 2013).

These geometric consistency assessments rank both elements and segments as either “Good”, “Fair” or “Poor”, so these analyses may produce several shortcomings (Hassan 2004), because:

1. Using only three group may not be sufficiently descriptive, because they do not consider those regions that are closer to excellence or extremely bad, for example “Very good” when $|V_{85} - V_d|$ is lower than 5 Km/h or “Very Poor” when $|V_{85} - V_d|$ is greater than 30 Km/h.

[2] GIS: Geographical Information System

2. There is no appreciable difference between 9.9 Km/h and 10.1 Km/h for $|V_{85} - V_d|$, and they are in different categories "Good" and "Fair", respectively (Watters and O'Mahony 2007).
3. There is no difference between variation of 9.9 Km/h for V_{85} of 50 Km/h (19.8%) or 100 Km/h (9.9%), consequently relative consideration could be presented.

According to the above and considering the (Bertrand-Vaschy-Buckingham Π –theorem) (Buckingham 1914), it is proposed to develop a new geometric design consistency model which would be multi-criteria and dimensionless.

In the development of this model, the criteria to be considered have been extracted from the previous geometric design consistency models, e.g. (Lamm, Beck, et al. 2007), and different risk-based models (e.g. (Austroads 2015)). The criteria to be considered are divided into the following 4 groups:

- Operating Speed;
- Vehicle Stability;
- Alignment Indices;
- Driver Workload.

Thus, the risk model which is developed in this project uses seven risk criteria, shown in Table 2.5, to quantify to the overall risk as shown in Figure 2.7.

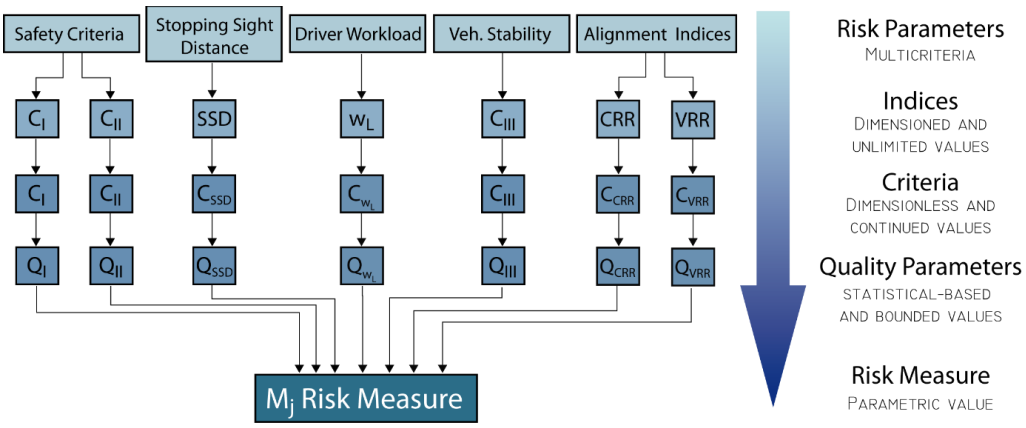


Figure 2.7 - Methodology for Risk Quantification

Table 2.5 - Risk Criteria

C_I	=	$\lvert V_{85_i} - V_d \rvert / V_d$	Design Speed risk parameter
C_{II}	=	$\lvert V_{85_i} - V_{85_{i-1}} \rvert / \emptyset V_{85}$	Operating Speed risk parameter
C_{III}	=	$e^{-\Delta f_R}$	Side Friction risk parameter
C_{SSD}	=	SSD_i / SSD_{V_d}	Stopping Sight Distance risk parameter
C_{CRR}	=	$\frac{R_i}{R_{avg}}$	Horizontal Alignment risk parameter
C_{VRR}	=	$\frac{\frac{L_i}{\lvert A \rvert}}{VR_{avg}}$	Vertical Alignment risk parameter
C_{WL}	=	$\mathrm{U} \cdot \mathrm{E} \cdot \mathrm{S} \cdot \mathrm{R}_f + \mathrm{C} \cdot \mathrm{WL}_{i-1}$	Driver Workload risk parameter

Each of these parameters is derived from its own equation or set of equations; these are described further in Technical Note TN11 – Model Description.

2.4 Quality Ranges

To facilitate easier interpretation, the risk parameters calculated by the risk model are normalised to produce “quality parameters” between 0.0 and 1.0, where 0.0 represents the lowest risk and 1.0 the highest, as shown in Figure 2.8.

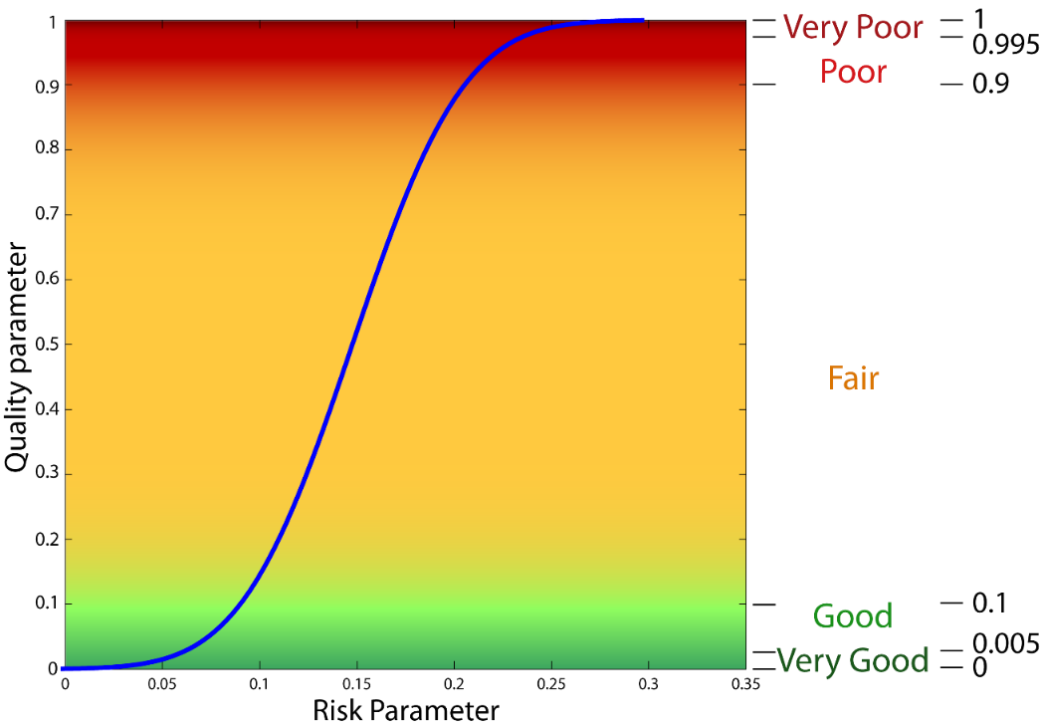


Figure 2.8 - Quality parameter estimation

This normalisation is achieved by fitting the calculated risk parameters to the cumulative distribution function (CDF) of the normal distribution, see Figure 2.8.

The result of this normalisation process is a set of seven quality parameters (Q_i), with each quality parameter corresponding to one of the risk criteria (C_i) in Table 2.6.

Table 2.6 - Risk Quality equations, criteria and indices of Risk analysis model

Parameter	*Indice* I_j		*Criterion* C_j		*Quality Equation* Q_j	
Operating Speed (General variation)	I_I	$\lvert V_{85_i} - V_d \rvert$	C_I	$\lvert V_{85_i} - V_d \rvert / V_d$	Q_I	$CDF(C_I, {}^{15}/_{90}, {}^{3.9}/_{90})$
Operating Speed (subsequently variation)	I_{II}	$\lvert V_{85_i} - V_{85_{i+1}} \rvert$	C_{II}	$\lvert V_{85_i} - V_{85_{i+1}} \rvert / \emptyset V_{85}$	Q_{II}	$CDF(C_{II}, {}^{15}/_{90}, {}^{3.9}/_{90})$
Vehicle Stability	I_{III}	Δf_R	C_{III}	$e^{-\Delta f_R} / f_{R_A}$	Q	$CDF(C_{III}, {}^{e^{0.015}}/_{0.1575}, 0.0195 \cdot e^{0.015} / 0.1575)$
Sight Distance	I_{SSD}	SSD	C_{SSD}	SSD_i / SSD_{V_d}	Q_{SSD}	$1 - CDF(C_{SSD}, 1.25, 0.2)$
Horizontal Alignment	I_{CRR}	CRR_i	C_{CRR}	$CRR_i = {}^{R_i}/_{R_{avg}}$	Q_{CRR}	$1 - CDF(C_{CRR}, 0.9, 0.12)$
Vertical Alignment	I_{VRR}	VRR_i	C_{VRR}	$VRR_i = {}^{\frac{L_i}{\lvert A \rvert}}/_{VR_{avg}}$	Q_{VRR}	$1 - CDF(C_{VRR}, 0.9, 0.12)$
Driver Workload	I_{WL}	WL	C_{WL}	$WL = U \cdot E \cdot S \cdot R_f + C \cdot WL_{i-1}$	Q_{WL}	$CDF(C_{WL}, 3, 0.875)$

The overall risk model which has been developed consists of a multi-criteria dimensionless model, which evaluates each road alignment element with a risk value M_i based on these quality parameters with the following formula:

$$M_i = w_1 \cdot Q_{C_{I_i}} + w_2 \cdot Q_{C_{II_i}} + w_3 \cdot Q_{C_{III_i}} + w_4 \cdot Q_{SSD_i} + w_5 \cdot Q_{CRR_i} + w_6 \cdot Q_{VRR_i} + w_7 \cdot Q_{Wl}$$

In this equation each of the quality parameters Q_i values is weighted according to the values w_i (where the weightings were derived from analysis of collision records for the pilot sites).

Normalisation of the quality parameters also allows categorisation of the risk and quality parameters according to discrete quality ranges, as shown in Table 2.7.

Table 2.7 – Quality Ranges

Range		Quality
0	0.005	Very Good
0.005	0.1	Good
0.1	0.9	Fair
0.9	0.995	Poor
0.995	1	Very Poor

2.5 Correlation of Risk Parameters with Collision Data

Finally, to determine properly the parameters weight, w_i, it is convenient to correlate the risk parameter with the collision data, this process is referred to as the weighting parameters calibration. In conducting this exercise, Road Safety Authority collision data from 2009 to 2013 at the Pilot Sites was employed and collated with the quality risk criteria along these alignments. The process was as follows:

1. *Allocate the collision data* along each alignment element (excluding intersections).

2. *"Translate" the collision occurrence to an ENSI value* (Expected Value of Severe Incident), Figure 2.9, which is a collision-risk relation measurement that considers the collision occurrences as a relative value of severe incidents, (Grande, et al. 2017) as follows:

$$ENSI_i = Fatal_i + {}^{1}/_{a} \cdot \mathrm{Serious}_i + {}^{1}/_{b} \cdot \mathrm{Minor}_i$$

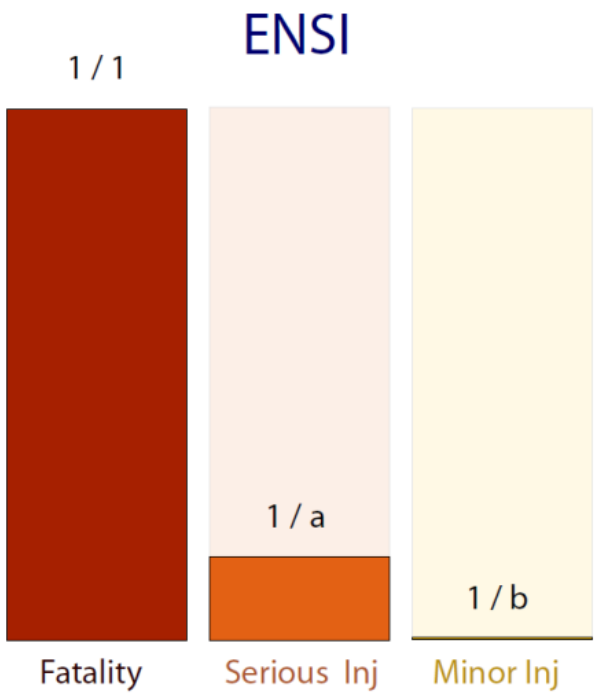


Figure 2.9 – ENSI Value Relationship

For this preliminary study, where only a subset of the network is analysed, four different ENSI scenarios have been considered. There parameters are:

Table 2.9 - ENSI fatal collision relationship parameters

ENSI Fatal Relation	Case			
	A	B	C	D
a	10	10	6.25	6.25
b	50	20	50	20

3. *Redefine the collision values (ENSI)* to obtain a relative value which may be correlated with the risk parameters. Thus, the ENSI value of each of the critical locations is recalculated as *Hazard value,* H_{val_i}, with the following expression:

$$H_{val_i} = \frac{ENSI_i}{AADT_i * Length_i}$$

Where:

AADT Annual Average Daily Traffic;

Length Length of Alignment Element.

4. *Define the quality parameters* in the forward and reverse directions for each point, using:

$$\mathrm{Q_{j_i}} = \frac{\mathrm{Q_{j_{i_{forward}}}} + \mathrm{Q_{j_{i_{reverse}}}}}{2}$$

5. *Obtain the weight values* using multilinear regression on the following formula:

$$H_{val_i} \cong H'_{val_i}$$

Where the hazard value obtained from ENSI values, H_{val_i} , should be equal to that obtained from risk parameters, H'_{val_i}:

$$H'_{val_i} = w'_1 \cdot Q_{C_{I_i}} + w'_2 \cdot Q_{C_{II_i}} + w'_3 \cdot Q_{C_{III_i}} + w'_4 \cdot Q_{SSD_i} + \\ + w'_5 \cdot Q_{CRR_i} + w'_6 \cdot Q_{VRR_i} + w'_7 \cdot Q_{Wl}$$

Where:

H'_{val_i} = Hazard value in element *i* from quality parameters;

w'_j = Weighted of Criterion *j;*

Q_{j_i} = Risk parameter value of criterion j in element *i.*

Consequently, the assumed weights come from the mean of average weights of mean regression values and the average weights of max regression values, resulting in:

Table 2.8 – Calibrated Risk Model Weighting Values

Weight	W_I	W_{II}	W_{III}	W_{ssd}	W_{ccr}	W_{vrr}	W_{wl}	**Sum**
	0.232	0.161	0.198	0.143	0.079	0.015	0.172	1

3. RISK ANALYSIS PROCESS

The developed risk analysis process comprises three stages: firstly, the geometric parameters of the road are derived from GPS (Global Positioning System) survey data, using a geometry definition model. This model also estimates the forward sight distance along the road. Secondly, an operating speed model is used to estimate vehicle operating speeds, based on the geometric data. Finally, the risk model combines the output of the first two models with various other data to produce a risk value for each road alignment element, Figure 3.1.

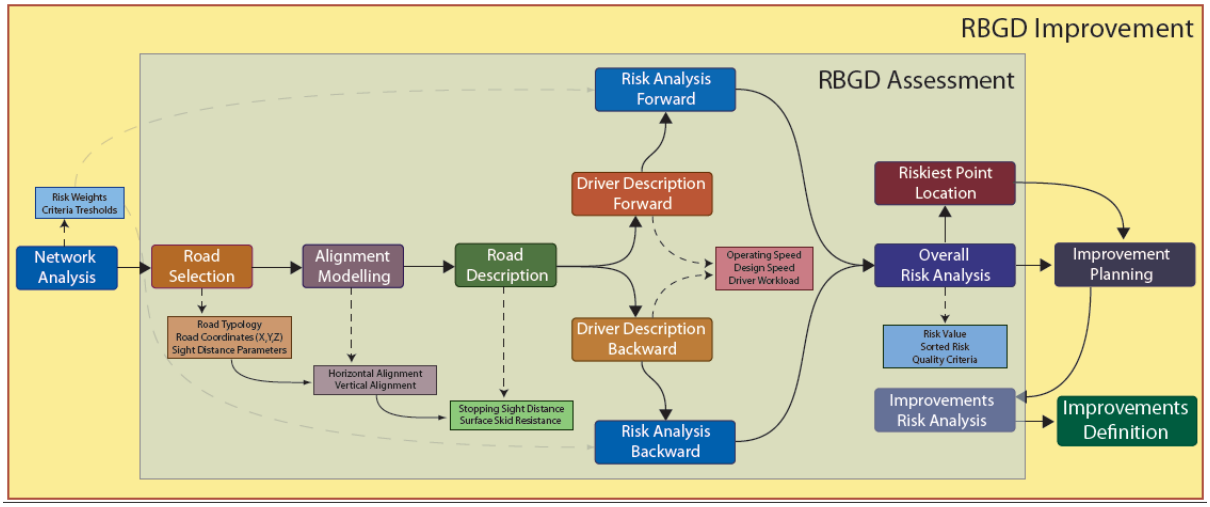


Figure 3.1 – Developed Risk Analysis Process

3.1 Geometry Definition Model

A geometry definition model has been developed to derive the geometric parameters of each road, based on GPS data. The GPS data used has been collected by Pavement Management Surveys, Ltd. as part of TII's annual road pavement surveys.

The survey data consists of a series of points recorded at roughly 5-metre centres along each lane of each road. The data points was converted to Irish Transverse Mercator (ITM) coordinates and processed to remove roundabout loops, duplicate points and GPS signal errors. As the GPS tracks for each route are typically recorded as part of multiple individual surveys, consecutive tracks needed to be smoothly joined together to form one single track in each direction along each road. The resulting tracks were then averaged to derive a single road centreline for each road. Speed-controlled areas (e.g. through towns and villages) and non-single carriageway roads were also removed from consideration, as the safety considerations for these roads are governed by different factors.

The resulting averaged track data consisted of points at 5-metres centres similarly to the GPS data from which it was derived. This averaged data was then processed using a geometry definition model to derive parameters such as tangent and curve lengths, horizontal curve radius, vertical grade and K-values for crest and sag curves.

The geometry definition model which was used considers the variation in azimuth (bearing) between consecutive points to determine the extents and parameters of horizontal tangents, curves and transitions along the road. Similarly, the variations of grade are analysed to determine the extents and parameters of vertical tangents, crest curves and sag curves.

Incorporated as part of the geometry definition model, a Stopping Sight Distance (SSD) model calculates the SSD along the road, based on assumed widths for the carriageway and verge.

The developed process flow is illustrated in Figure 3.2.

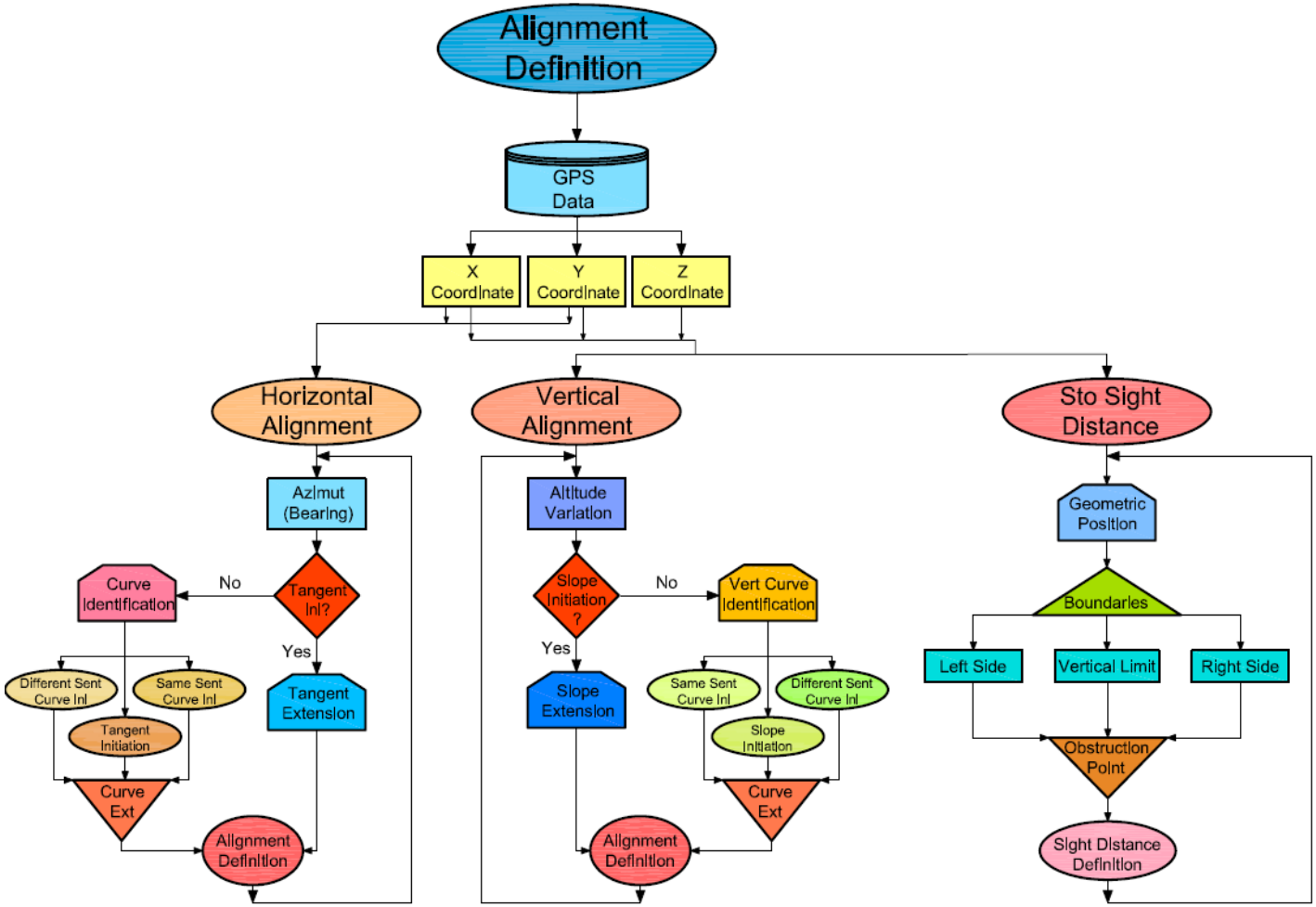


Figure 3.2 – Alignment and SSD Definition Process

3.2 Operating Speed Model

An operating speed model has been developed which is used to estimate the operating speed (i.e. the 85th percentile speed) of vehicles for each road alignment element, as shown in Figure 3.3. The operating speed model calculates different values for curves, independent tangents and dependent tangents. “Independent tangents” are tangents where the tangents are assumed to be sufficiently long for vehicles to accelerate to their desired speed before having to decelerate on the approach to the following curve; dependent tangents are those tangents where the vehicles must start to decelerate before having reached their desired speed. The model considers each of these cases separately.

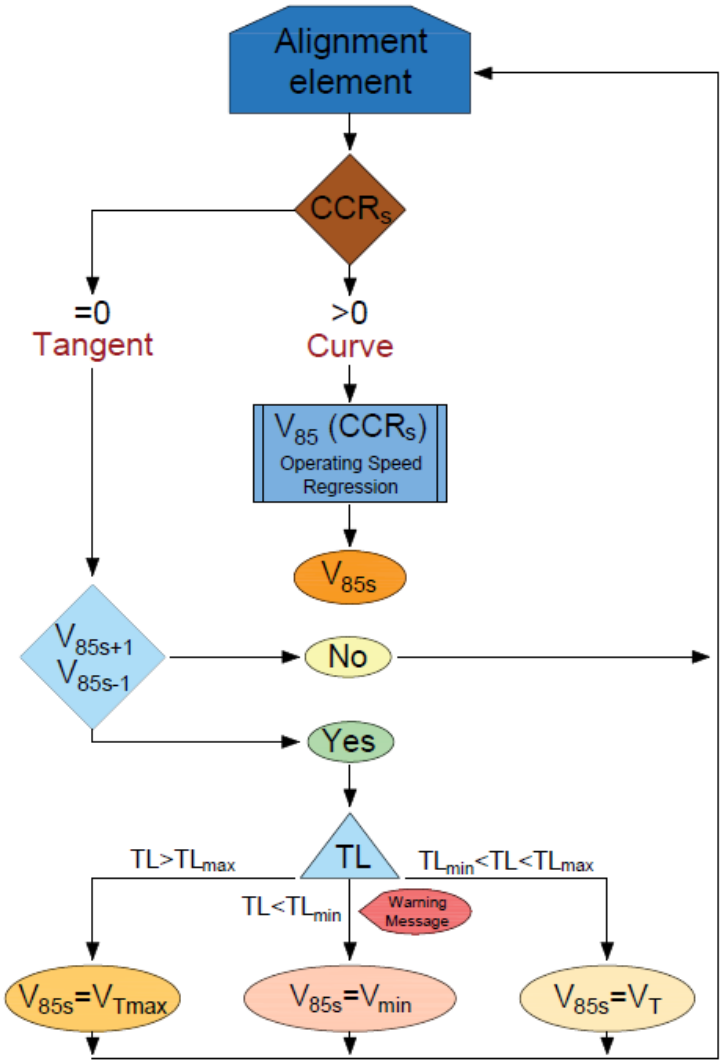


Figure 3.3 – Operating Speed Model

3.3 Risk Model

Once the geometric and SSD parameters have been established and the operating speed calculated, the resulting data is processed using the risk model to derive the overall risk measure for each alignment element, for both the forward and reverse directions. The definition of which direction is “forward” is arbitrary, but in general the

direction of increasing chainage in TII's network model has been assumed as "forward". A given alignment element will typically have different risk measures for the forward and reverse directions, as the operating speed of the vehicles is dependent on the direction of approach.

One benefit of the multi-criteria nature of the risk model is that in addition to producing an overall risk measure for each element, the individual quality parameters can also be examined to study which parameters make the greatest contribution to the risk for the element in question. In other words, rather than simply suggesting that remedial measures might be appropriate at a particular location, examination of the model results can give insight into what specific kind of intervention might be optimal. For example, if the largest contribution to the risk measure is from insufficient side friction, then this could potentially be remediated without the need for a realignment. Conversely, if side friction were found to have minor influence on the risk measure then a resurfacing strategy can be shown to be of little benefit. Similarly, if the vertical alignment is found to be a significant factor, a horizontal realignment of the road might not provide a worthwhile improvement in the overall risk.

A further benefit is that the model is not just applicable to existing roads: it can be equally well applied to proposed mitigation schemes. This provides the potential for the model to be used to examine various alternative proposals, and also enables the use of an iterative approach to find an optimal solution. Likewise, the model can also identify clusters of high-risk alignment elements which could be mitigated with a single improvement scheme.

Finally, the model also makes it possible to compare the alignment *consistency* of an existing alignment with proposed design options. This can be used to identify where a proposed design option might in fact lead to an increase in risk at the tie-ins to the existing alignment.

Examples of the practical application of the model are given in Section 5.

Ultimately an extremely powerful Asset Management Tool has been developed.

4. PILOT SITE ANALYSIS

Data from 30 pilot sites was used for the calibration/testing of the operating speed model and risk model. The pilot sites used are all locations with significant constraints of geometry or driver expectancy which may have contributed to collisions around those locations. In addition to their use for calibration, the Risk Based Geometric Design (RibGeom) methodology was used to model a section of each route (approximately 5km in length) with the aim of determining the risk range along each route in the vicinity of each pilot site.

Technical Papers were prepared for the pilot site inspections and assessments in 5 groups as listed in Appendix A.

In addition to the data available for the road network as a whole, further data was gathered for each of these pilot sites. As part of a related project, mobile LiDAR[3] surveys were carried out by Pavement Management Services, Ltd. (PMS). A team at Maynooth University (MU) then processed this LiDAR data to produce three-dimensional ground models. ROD then further processed these models using Autodesk Civil 3D to derive data on carriageway and verge widths which in turn were used to calculate accurate sight distance values for each route. These sight distance values were then used for the calibration of the SSD model.

Speed surveys were commissioned recording vehicle speeds at each of the pilot site locations. At each location, the speed of vehicles was recorded at two points: on the approach to the bend and at the bend itself. This allowed the deceleration of vehicles approaching the bend to be examined, which is a significant parameter in the causation of collisions. This speed survey data was used to calibrate the operating speed model.

Individual site inspections were also carried out for each of the sites to collect further information related to risk factors. While not all of these types of information were of a nature that could be incorporated into the risk model, they all served to provide a “sanity check” to ensure that the model was accurately reflecting the safety considerations on the ground. The types of information collected from these site inspections included:

a) Road and Verge Widths;
b) Stopping Sight Distance (SSD);
c) Drainage and relative ground levels;
d) Pavement Condition;
e) Road Markings and Traffic Signs;
f) Evidence of Traffic Collisions;
g) Accesses to adjoining lands;
h) Photographs.

The risk model was applied to each of these pilot sites and the results collated and analysed. The mean risk values were compared, and the standard deviations of the

[3] LiDAR: Light Detection and Ranging, a laser-based survey method.

risk values were used to examine variations of risk within each pilot site route. Significant variations of calculated risk were found between the various sites, as can be seen in Figure 4.1.

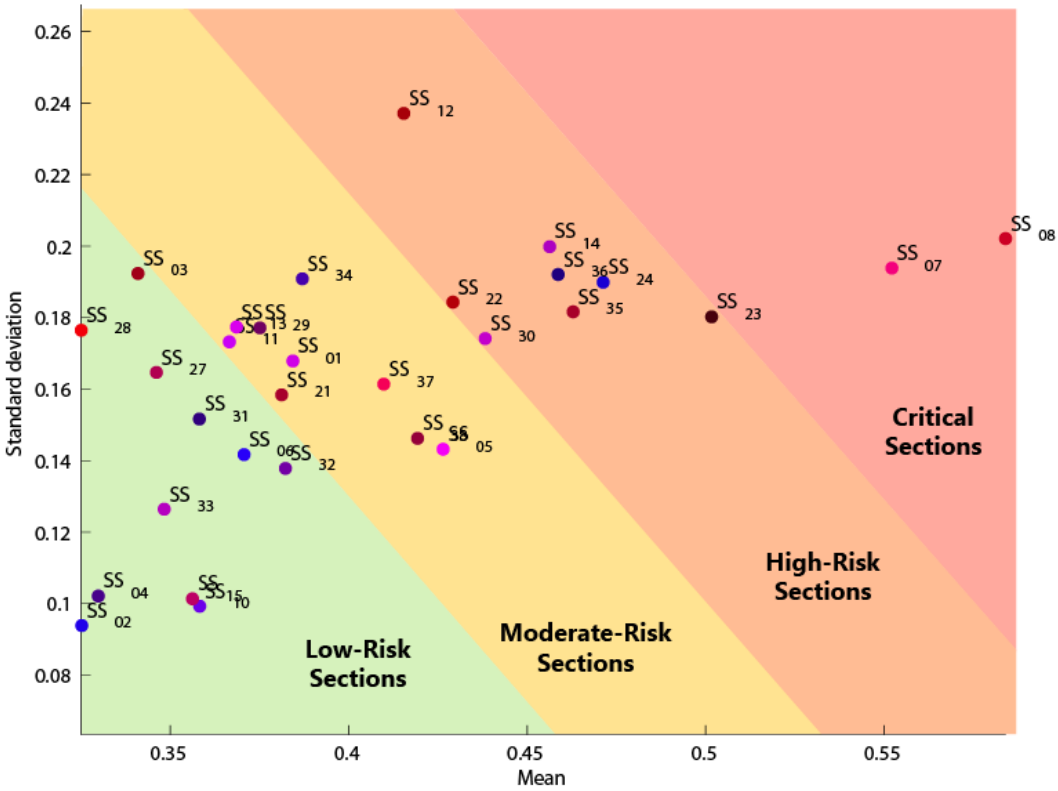


Figure 4.1 – Pilot Sites Risk Grouping

One important point to note is that, at present, the risk model does not allow for variations in traffic volumes. As such, it can be used to compare different parts of a given route with similar traffic volumes; it can also be used to compare multiple proposed design options for a specific location. This point is well illustrated by pilot sites SS_07 and SS_08 in Figure 4.1 – these two sites have the two highest mean risk values and also have among the highest standard deviations of risk values. However, these two sites are both located on the N67 in Co. Clare, which has the lowest traffic volumes of any of the routes studied. Incorporating the effects of traffic volumes is a priority for future improvement of the risk model.

It should also be noted that for logistical reasons, the pilot sites which were examined are located mainly in the western part of Ireland. If further pilot sites are to be examined in the future, it would be desirable to ensure a more representative geographical spread, as this would improve the accuracy of the method in its application to the road network as a whole.

5. CASE STUDIES

Case Studies were undertaken for 3 sections of typical national secondary routes that contained several of the pilot sites as described in Chapter 4 previously. Technical Papers were prepared for the case studies as listed in Appendix A.

5.1 N66 (former), County Galway, Case Study

5.1.1 Case Study Route Details

The N66 is a 23km length of single carriageway road between Loughrea and Gort in County Galway. The route passes primarily through rural countryside, but passes through the village of Kilchreest, Co Galway. The extents of the N66 study route are indicated on Figure 5.1. (In 2017 this route was reclassified as Regional Road R380).

Four of the pilot sites that were included in this Risk Based Geometric Design project are located on the N66. These are SS_05, SS_14, SS_15 and SS_22. Observations made on site indicate that the road width varies from 5.5m to 6.5m with 0.5m hard strips and typical verge widths of 2m, however verge widening has been provided at various locations along the route, primarily to improve forward sight visibility at bends.

The character of the road layout on the N66 between Loughrea and Gort varies widely along its length. The route is made up of sections of straight or nearly straight road interspersed with bends of varying radii. There are sections of the route which are bendier than others, particularly at Gortnamaken Bridge, east of Kilchreest, where the road narrows to a single lane shuttle crossing of the Duniry River, with restricted visibility on the approaches to the bridge.

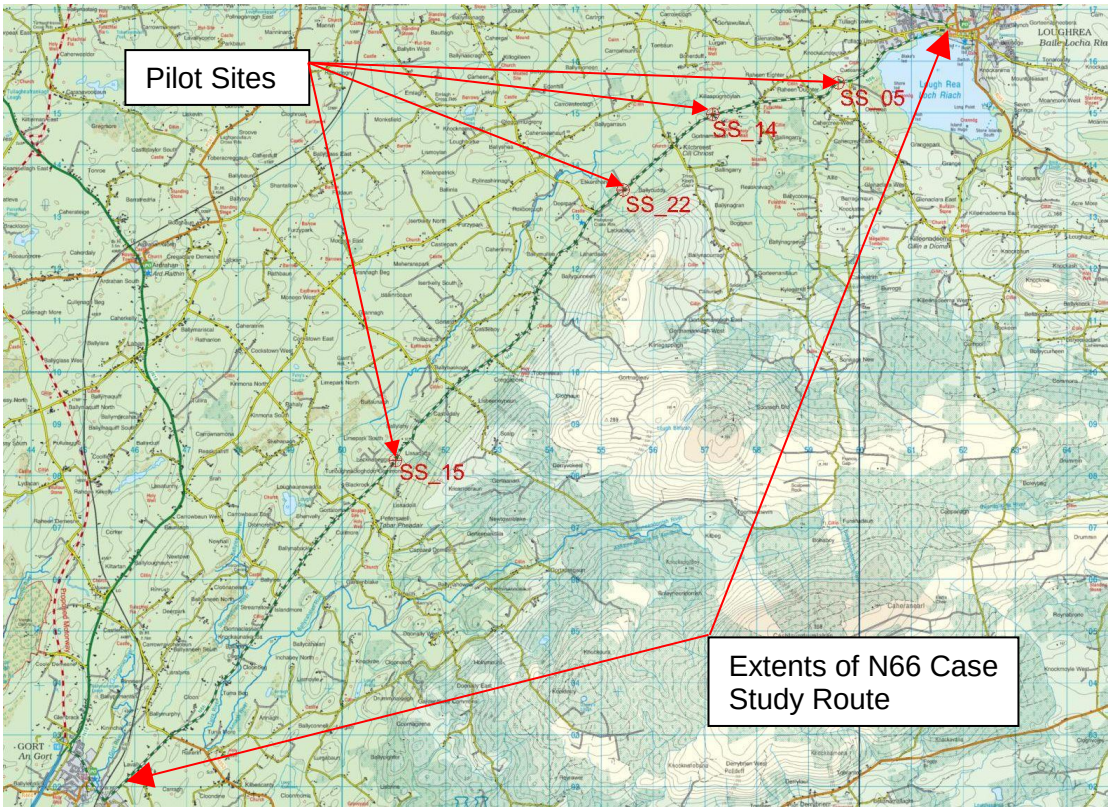


Figure 5.1 – N66 / R380 Case Study Route

5.1.2 Recorded Collision History

A review of the recorded collisions along the N66 indicates that the following collisions occurred:

1) Fatal Collisions – 1 No. South of Loughrea;
2) Serious Collisions – 3 No., 1 at Loughrea, 1 South of Kilchreest and 1 at Gort;
3) Minor Collisions – 12 No. spread along the route;
4) Material Damage Collisions – 51 No. spread along the route.

The collision data indicates that a series of 11 Material Damage Collisions occurred at a location approximately 9km south of Loughrea. This location coincides with a high-risk location identified by the Risk Model (Curve No.94).

The collision data also indicates that a series of 5 Material Damage Collisions occurred in the vicinity of Gortnamacken Bridge, approximately 5km south of Loughrea. This location also coincides with a high-risk location identified by the Risk Model (Curve No.48 to No.57).

The remainder of the recorded collisions appear to be randomly spread with no apparent collision clusters. The locations of recorded collisions on the N66 are indicated on Figure 5.2.

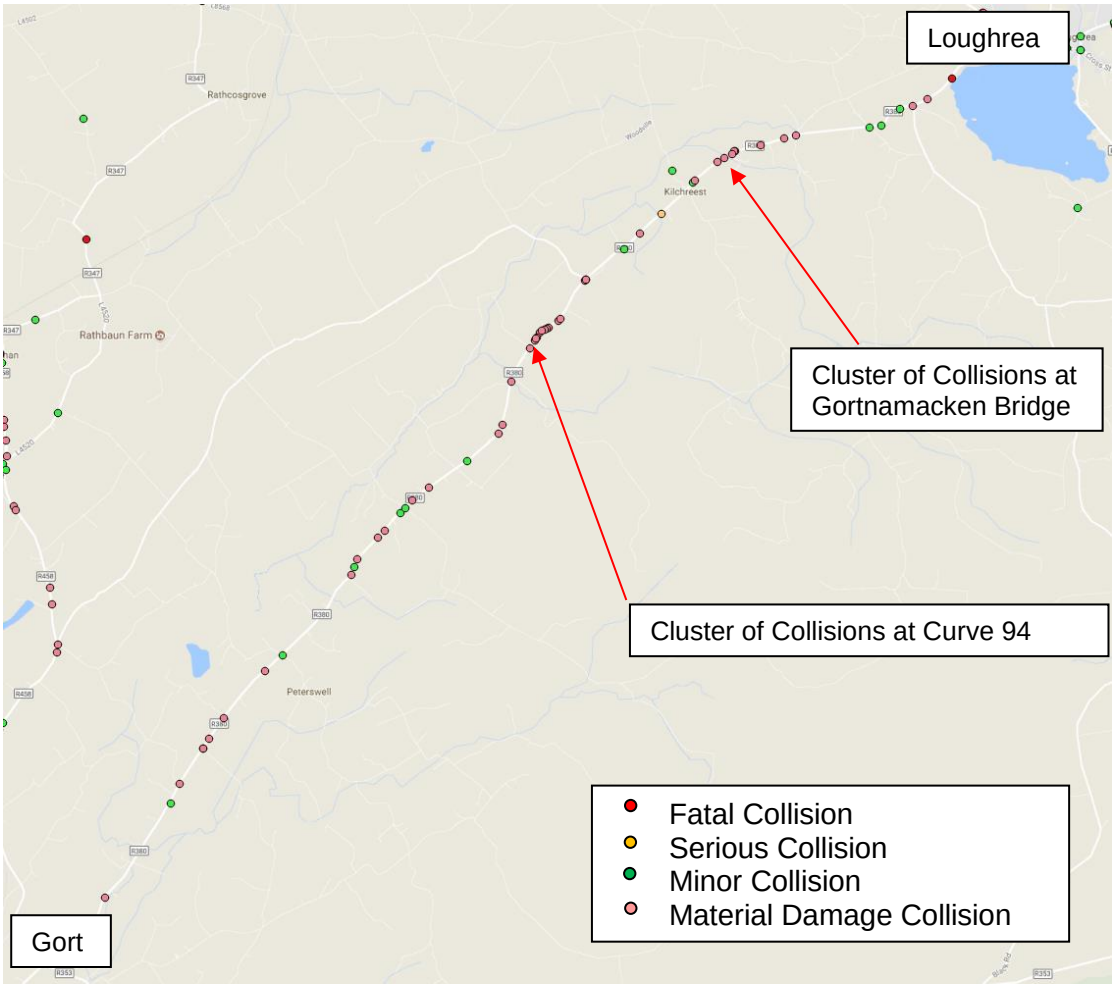


Figure 5.2 – N66 Collision History

5.1.3 Analysis of Existing N66 Route

Existing Alignment Derivation

The alignment derived for the N66 route indicates the presence of 129 horizontal curves along its length, with curve radii ranging from 40m to 4,360m. The horizontal curves can be grouped into bands as per Table 5.1.

Table 5.1 – N66 Existing Horizontal Alignment Curve Radii

Curve Radius	DN-GE0-03031 Standard for 100km/h Design Speed (Table 1.3)	Number of Curves
<127m	Beyond Standard	8 (6.2%)
127m - 180m	Beyond Standard	9 (7.0%)
180m - 255m	Four Steps Below Desirable Minimum	16 (12.4%)
255m - 360m	Three Steps Below Desirable Minimum	15 (11.6%)
360m – 510m	Two Steps Below Desirable Minimum	14 (10.9%)
510m – 720m	One Steps Below Desirable Minimum	21 (16.3%)
>720m	Desirable Minimum	46 (35.6%)

The analysis of the N66 route alignment indicates that 48 out of the 129 horizontal curves (37.2%), are below the requirements of Table 1.3 of TII Publication DN-GEO-03031 for a Design Speed of 100km/h on a Type 2 Single Carriageway road, although much of the route is more similar to a Type 3 Single Carriageway for which 4 steps relaxations would be permitted. This rate does not take into account the super-elevation requirements of Table 1.3 of DN-GEO-03031 for horizontal curves.

Overall Risk Rating for Existing Route

The Overall Risk Ratings computed by the model for each element of the existing N66 route range from a lowest Overall Risk Rating of 0.151 to a highest Overall Risk Rating of 0.953. Figure 5.3 shows the Overall Risk profile along the N66 route as determined by the Risk Model. Table 5.2 shows the top 15 Overall Risk Rated alignment elements for the route.

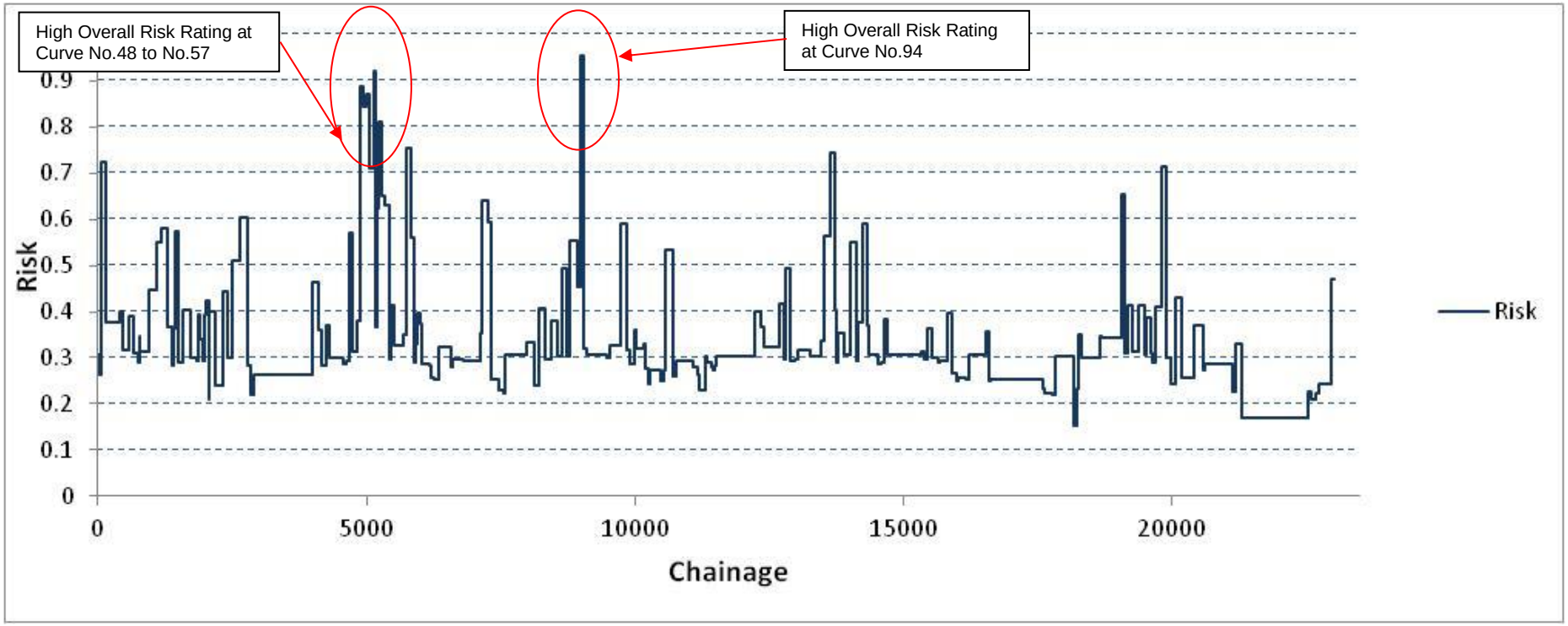


Figure 5.3 – N66 Existing Overall Risk Rating Profile

Table 5.2 – N66 Existing Top 15 Overall Risk Rated Elements

Risk Order	Alignment Element ID	Alignment Element Type	Start Chainage	End Chainage	Overall Risk
1	94	Bend	8984.095	9037.099	0.953
2	52	Bend	5131.071	5186.075	0.921
3	48	Bend	4881.028	4934.032	0.889
4	50	Bend	4986.039	5062.052	0.871
5	49	Bend	4934.032	4986.039	0.845
6	55	Bend	5241.076	5301.079	0.809
7	62	Bend	5753.083	5821.086	0.754
8	137	Bend	13651.110	13742.110	0.745
9	3	Bend	74.000	154.002	0.723
10	187	Bend	19821.120	19910.130	0.715
11	51	Bend	5062.052	5131.071	0.710
12	177	Bend	19064.120	19123.120	0.652
13	56	Bend	5301.079	5358.081	0.652
14	78	Bend	7163.088	7260.090	0.641
15	57	Bend	5358.081	5436.083	0.630

Table 5.2 indicates that the highest Overall Risk of 0.953 occurs at a single curve (Curve No.94) between Ch.8+984.095 and Ch.9+037.099, highlighted in yellow.

The second high risk section with ratings ranging from 0.921 to 0.630 is at a series of curves at Gortnamacken Bridge (Curve No.48 to No.57), highlighted in green.

Figure 5.4 – N66 at Gortnamacken Bridge

Operating Speed Profile for N66

The Operating Speed Profile shown in Figure 5.5 indicates that the operating speed along the existing N66 route ranges from a high of 98 km/h to a low of 28 km/h, and the Average Operating Speed is 87km/h. This equates to an unusually wide speed variation along the route of 70km/h. In accordance with the *Geometric Design Consistency Model* developed by Lamm et al. (1999), a speed range in excess of 20 km/h indicates a Consistency Rating of Poor. As part of the Risk Based Geometric Design project, it has been agreed with TII to add an additional Consistency Rating of Very Poor, where the speed differential is in excess of 30km/h. The operating speed profile on the N66 route falls far outside into this band.

Figure 5.5 indicates that the two locations where the greatest speed variation occurs coincide with the two locations where the Risk Model determined the highest risk occurs.

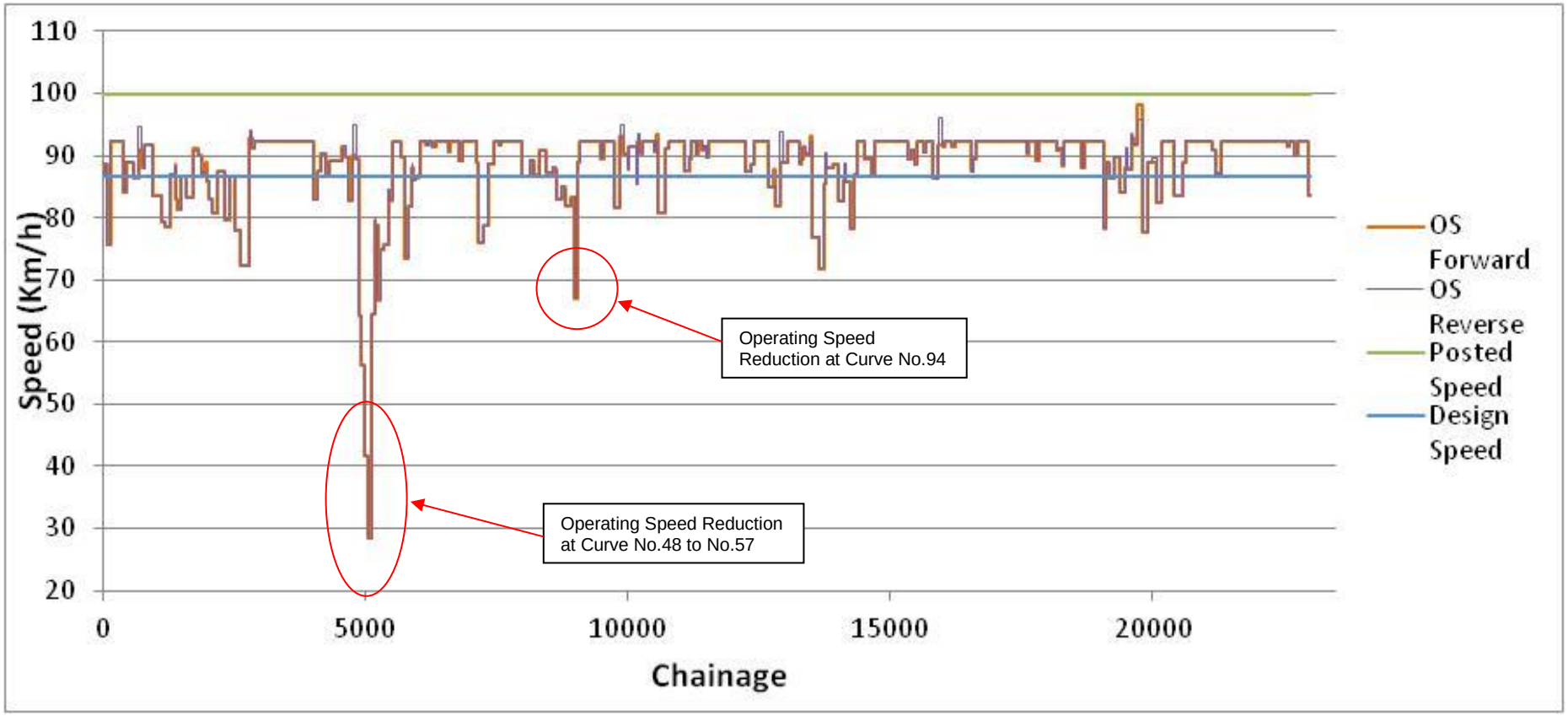


Figure 5.5 – N66 Existing Operating Speed Profile

Risk Analysis of Existing Curve No. 94

The highest Overall Risk Rating of 0.953 was observed at alignment element Curve No. 94, a short single 105m radius curve located between Ch.8+984 and Ch.9+037, highlighted in yellow in Table 5.3. This curve coincides with a cluster of collisions. Table 5.3 shows the alignment elements on either side of Curve No.94. The Overall Risk Rating associated with Curve 94 is significantly higher than the Overall Risk Rating for the adjacent alignment elements.

Table 5.3 – Existing N66 Overall Risk Ratings for Curve 94 and Approaches

Alignment Element ID	Alignment Element Type	Start Chainage	End Chainage	Curve Radius (m)	Overall Risk Rating	Risk Order
86	Bend	8211.592	8325.092	468	0.406	45
87	Tangent	8325.092	8435.092		0.297	129
88	Bend	8435.092	8571.093	481	0.381	58
89	Tangent	8571.093	8639.593		0.323	111
90	Bend	8639.593	8742.093	263	0.493	32
91	Tangent	8742.093	8800.093		0.304	113
92	Bend	8800.093	8931.095	235	0.550	26
93	Bend	8931.095	8984.095	318	0.453	35
94	Bend	8984.095	9037.099	105	0.953	1
95	Bend	9037.099	9094.099	808	0.319	91
96	Tangent	9094.099	9483.600		0.306	108
97	Bend	9483.600	9533.100	746	0.300	122
98	Tangent	9533.100	9740.600		0.326	88
99	Bend	9740.600	9851.101	230	0.590	20
100	Tangent	9851.101	9921.101		0.316	94

Risk Analysis of Existing Curves No. 48 to No.57 at Gortnamacken Bridge

The Risk Model identified a series of curves with Overall Risk Ratings of between 0.921 and 0.630 between Ch.4+881.0258 and Ch.5+436.083 at Gortnamaken Bridge. Alignment Elements No.48 to No.57 correspond to a series curves with radii ranging from 40m to 142m as highlighted in green in Table 5.4. This series of curves coincides with a cluster of Material Damage Collisions indicated by the collision data. The Overall Risk Ratings associated with Curve No.48 to No.57 are significantly higher than the Overall Risk Rating for the adjacent alignment elements.

Table 5.4 – Existing N66 Overall Risk Ratings for Element No. 40 to No. 60

Alignment Element ID	Alignment Element Type	Start Chainage	End Chainage	Curve Radius (m)	Overall Risk Rating	Risk Order
40	Tangent	4153.027	4249.527		0.283	157
41	Bend	4249.527	4305.027	502	0.369	64
42	Bend	4305.027	4555.027	956	0.300	121
43	Tangent	4555.027	4631.027		0.288	151
44	Bend	4631.027	4671.027	848	0.292	139
45	Bend	4671.027	4753.028	253	0.569	23
46	Tangent	4753.028	4832.028		0.312	99
47	Bend	4832.028	4881.028	857	0.379	59
48	Bend	4881.028	4934.032	89	0.889	3
49	Bend	4934.032	4986.039	70	0.845	5
50	Bend	4986.039	5062.052	43	0.871	4
51	Bend	5062.052	5131.071	40	0.710	11
52	Bend	5131.071	5186.075	89	0.921	2
53	Tangent	5186.075	5193.575		0.366	68
54	Bend	5193.575	5241.076	189	0.624	16
55	Bend	5241.076	5301.079	98	0.809	6
56	Bend	5301.079	5358.081	142	0.652	13
57	Bend	5358.081	5436.083	143	0.630	15
58	Tangent	5436.083	5454.083		0.295	132
59	Bend	5454.083	5526.083	294	0.414	41
60	Tangent	5526.083	5676.583		0.327	87

5.1.4 Potential Realignments at High Collision Risk Sections

Indicative realignment schemes were developed for the two highest collision risks, Curve No.94 and Curves No.48 to 57 at Gortnamacken Bridge. These indicative realignment schemes comprised Option 1 for a Desirable Minimum Standard and Option 2 with Relaxations of 3 or 4 steps.

5.1.5 Analysis of Curve 94 Indicative Realignments

Curve No. 94 Option 1

An indicative realignment scheme was developed at this location using a DN-GEO-03031 compliant alignment, using 720m radii curves. A curve radius of 720m equates to a Desirable Minimum radius curve with super-elevation of 5%, in accordance with Table 1.3 of DN-GEO-03031.The realignment of the N66 using a compliant alignment at this location would require the realignment of 0.8 km of road and would likely require the acquisition of two houses. The indicative realignment is shown on Figure 5.6.

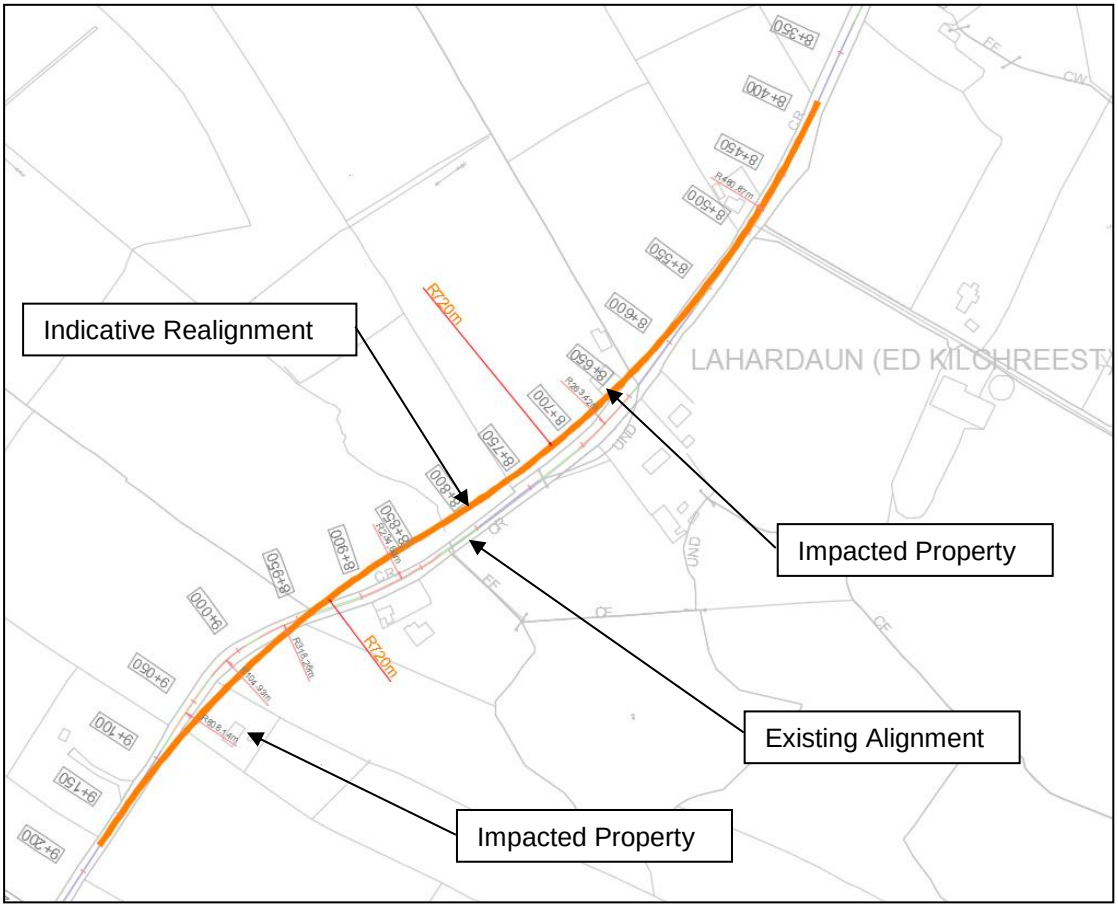


Figure 5.6 – N66 Curve 94 Option 1 Alignment

The Risk Model was re-run using this realignment option to demonstrate the impact on the Overall Risk Rating of Curve No.94 and the adjacent curves. The results of this analysis are shown in Table 5.5.

Table 5.5 – Option 1 Realignment Risk Ratings for Element No. 86 to No. 100

Alignment Element ID	Alignment Element Type	Curve Radius/m	Overall Risk Rating	Risk Order
86	Bend	468	0.331	63
87	Tangent		0.207	179
88, 89, 90	Bend	720	0.164	182
91,92,93,94,95	Bend	720	0.161	183
96	Tangent		0.215	176
97	Bend	746	0.273	136
98	Tangent		0.309	82
99	Bend	230	0.592	11
100	Tangent		0.310	79

Curve No.94 Option 2 Realignment

A second realignment scheme option would involve the replacement of Curve Nos. 93, 94 & 95 with a 255m radius curve, which is a 3 Steps Relaxation below Desirable Minimum. This option would require the realignment of 0.4 km of road, which is half the length of the higher standard alternative and would require the acquisition of much smaller land parcels from adjoining properties. The indicative reduced standard realignment at Curve No. 94 is shown on Figure 5.7.

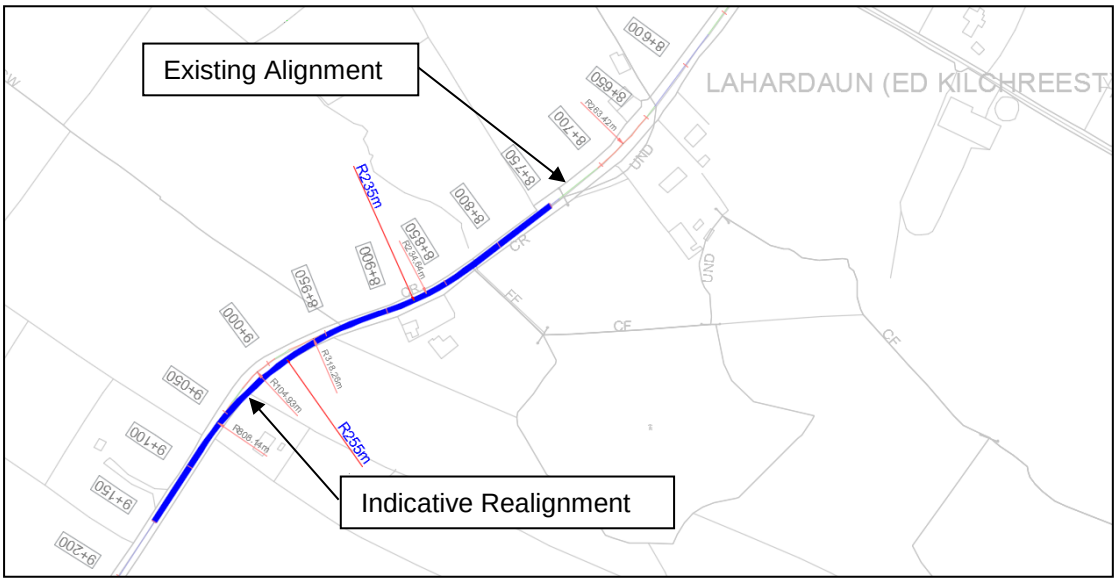


Figure 5.7 – N66 Curve 94 Option 2 Realignment

The Risk Model was re-run using this indicative reduced standard alignment to demonstrate the impact on the Overall Risk Rating of Curve No.94 and the adjacent curves. The results of this analysis are shown in Table 5.6.

Table 5.6 – Curve 94 Option 2 Realignment Risk Ratings

Alignment Element ID	Alignment Element	Curve Radius/m	Risk Rating	Risk Order
86	Bend	468	0.401	42
87	Tangent		0.265	154
88	Bend	481	0.370	61
89	Tangent		0.249	165
90	Bend	269	0.433	32
91	Tangent		0.223	185
92	Bend	235	0.457	26
93,94,95	Bend	255	0.428	34
96	Tangent		0.314	90
97	Bend	746	0.317	83
98	Tangent		0.305	105
99	Bend	230	0.591	13
100	Tangent		0.306	102

Curve No.94 Indicative Realignment Comparison

Table 5.7 – Comparison of N66 Alignment Risk Ratings for Options

				Option 1		Option 2	
Alignment Element ID	Alignment Element Type	Existing Alignment		Desirable Minimum		3 Steps Relaxations	
		Overall Risk	Risk Order	Overall Risk	Risk Order	Overall Risk	Risk Order
86	Bend	0.406	45	0.331	63	0.401	42
87	Tangent	0.297	129	0.207	179	0.265	154
88	Bend	0.381	58	0.164	182	0.370	61
89	Tangent	0.323	111			0.249	165
90	Bend	0.493	32			0.433	32
91	Tangent	0.304	113	0.161	183	0.223	185
92	Bend	0.550	26			0.457	26
93	Bend	0.453	35			0.428	34
94	Bend	0.953	1				
95	Bend	0.319	91				
96	Tangent	0.306	108	0.215	176	0.314	90
97	Bend	0.300	122	0.273	136	0.317	83
98	Tangent	0.326	88	0.309	82	0.305	105
99	Bend	0.590	20	0.592	11	0.591	13
100	Tangent	0.316	94	0.310	79	0.306	102

Table 5.7 shows a comparison of the Overall Risk Rating at Curve No.94 for the existing alignment, a Desirable Minimum standard realignment, and an option with 3

steps Relaxations. The Overall Risk at Curve No.94 reduces to 0.161 for the higher standard realignment, and to 0.428 for the reduced standard realignment. However, the risk rating for the alignment with 3 steps relaxations is more consistent with the rest of the existing road, and therefore achieves sufficient improvement through a scheme that is only half as long as the higher standard option, and that is less likely to result in risk migration to adjoining unimproved sections of road.

5.1.6 Realignment Options at Gortnamacken Bridge

Curve No. 48 to 57 Option 1

An indicative realignment scheme as shown in Figure 5.8 was developed at this location using a DN-GEO-03031 desirable minimum compliant alignment, using a 720m radius curve. This option would require the realignment of 0.8 km of road and the likely acquisition of two houses.

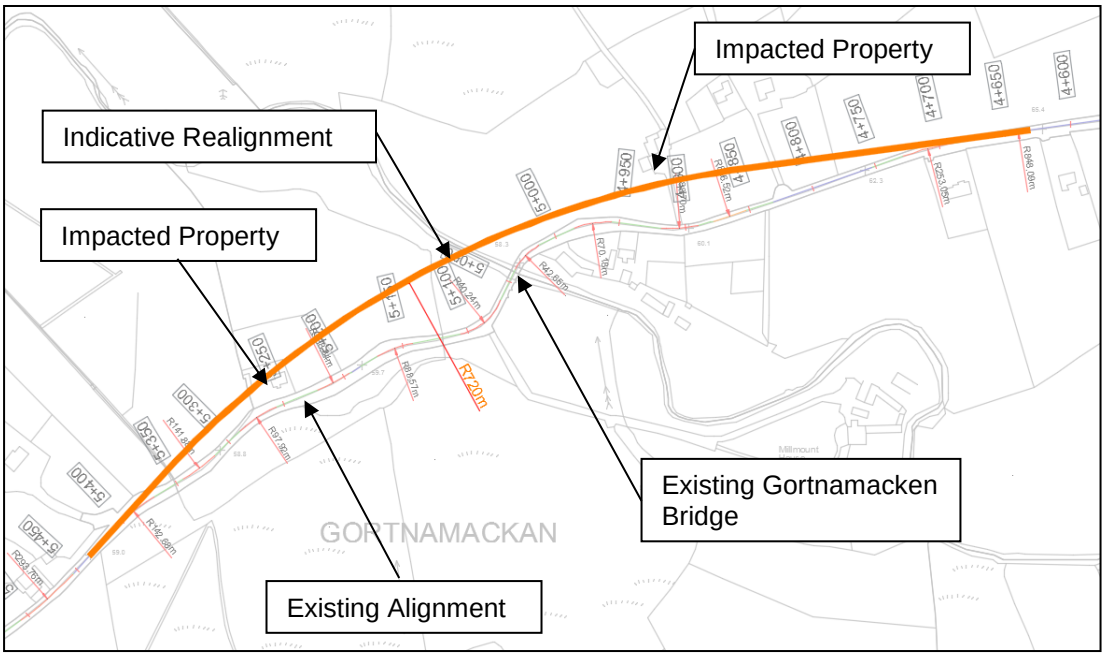


Figure 5.8 – N66 Curve 48 to 57 Option 1 Alignment

The Risk Model was re-run using this indicative compliant alignment to demonstrate the impact on the Overall Risk Rating of Curve No.48 to No.57 and the adjacent curves. The results of this analysis are shown in Table 5.8.

Table 5.8 – Risk Ratings for Option 1 Realignment at Gortnamacken Bridge

Alignment Element ID	Alignment Element Type	Curve Radius/m	Overall Risk Rating	Risk Order
40	Tangent		0.214	177
41	Bend	502	0.291	123
42	Bend	956	0.205	180
43	Tangent		0.215	174
44 to 57	Bend	720	0.237	159
58	Tangent		0.224	170
59	Bend	294	0.436	24
60	Tangent		0.327	66

Curve No. 48 to No.57 Option 2

The second indicative realignment scheme involved a series of 180m and 255m radius curves at 3 and 4 steps relaxations with realignment of 0.6 km of road and no impact for houses as shown in Figure 5.9.

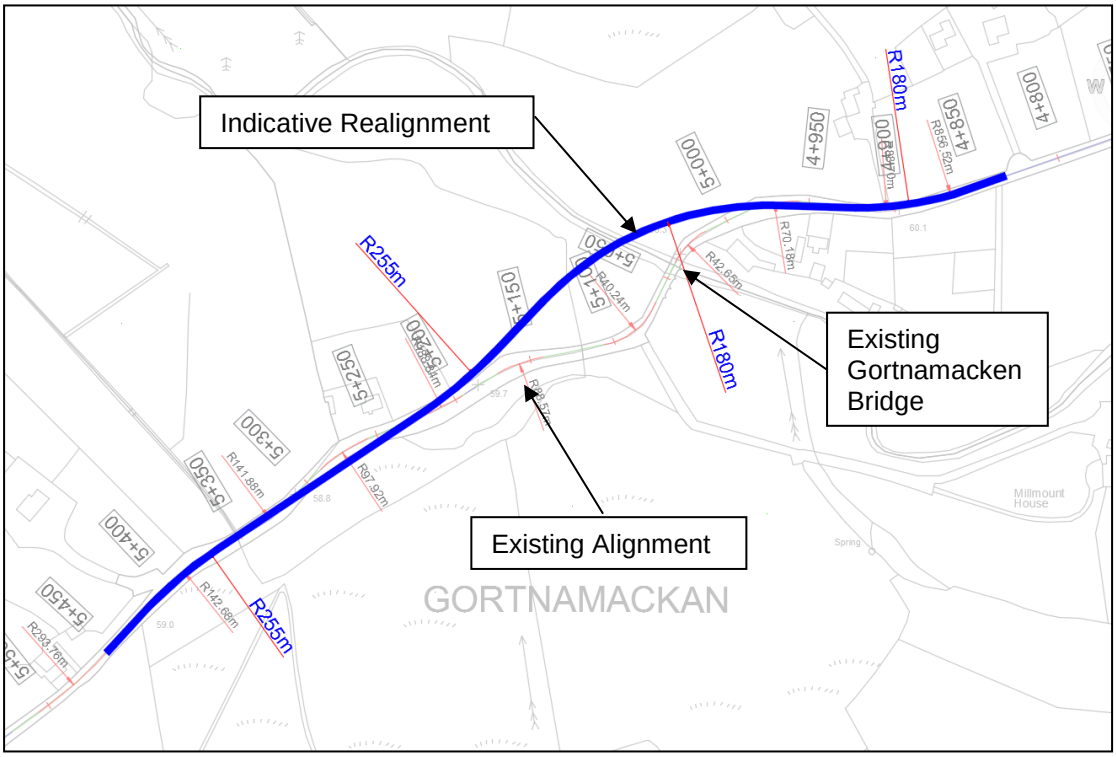


Figure 5.9 – Option 2 Realignment at Gortnamacken Bridge

This indicative reduced standard alignment was re-analysed using the Risk Model, which indicated that the Overall Risk Rating for the same section of road changed to the values indicated in Table 5.9.

Table 5.9 – Risk Ratings for Option 2 at Gortnamacken Bridge

Alignment Element ID	Alignment Element Type	Curve Radius/m	Overall Risk Rating	Risk Order
40	Tangent		0.291	134
41	Bend	502	0.368	64
42	Bend	956	0.297	109
43	Tangent		0.290	136
44	Bend	940	0.295	115
45	Bend	252	0.540	18
46	Tangent		0.314	88
47+48	Bend	180	0.669	6
49 to 52	Bend	180	0.505	23
53	Tangent		0.221	188
54 to 56	Bend	255	0.509	21
57+58	Bend	255	0.521	19
59	Bend	287	0.446	29
60	Tangent		0.314	87

Curve No. 48 to No.57 Indicative Realignment Comparison

Table 5.10 – Comparison of Options at Gortnamacken Bridge

Alignment Element ID	Alignment Element Type	Existing Alignment		Option 1 Des. Minimum		Option 2 Relaxations	
		Risk	Rank	Risk	Rank	Risk	Rank
40	Tangent	0.283	157	0.214	177	0.291	134
41	Bend	0.369	64	0.291	123	0.368	64
42	Bend	0.300	121	0.205	180	0.297	109
43	Tangent	0.288	151	0.215	174	0.290	136
44	Bend	0.292	139	0.237	159	0.295	115
45	Bend	0.569	23			0.540	18
46	Tangent	0.312	99			0.314	88
47	Bend	0.379	59			0.669	6
48	Bend	0.889	3				
49	Bend	0.845	5			0.505	23
50	Bend	0.871	4				
51	Bend	0.710	11				
52	Bend	0.921	2				
53	Tangent	0.366	68			0.221	188
54	Bend	0.624	16			0.509	21
55	Bend	0.809	6				
56	Bend	0.652	13				
57	Bend	0.630	15			0.521	19
58	Tangent	0.295	132	0.224	170		
59	Bend	0.414	41	0.436	24	0.446	29
60	Tangent	0.327	87	0.327	66	0.314	87

Table 5.10 shows a comparison of the risk reductions for both options at Gortnamacken Bridge and indicates that Option 1 provides the most consistent risk overall in the context of the adjacent elements and would therefore be the better option. In this location there is a quite poor alignment over almost 1 km length of the route, and the shorter 0.6 km long Option 2 would still retain some relatively sharp bends which limits the risk reduction benefits.

5.1.7 N66 Alignment Assessment Findings

Overall Risk Reduction

On the basis of the assessment of the existing N66 alignment and the indicative realignments developed at the two highest risk locations, the Risk Model was re-run using the 2 preferred realignment options to assess the combined benefits in the context of the overall route. Figure 5.10 shows a comparison of the before and after Risk Profiles once the two highest risk sections are improved.

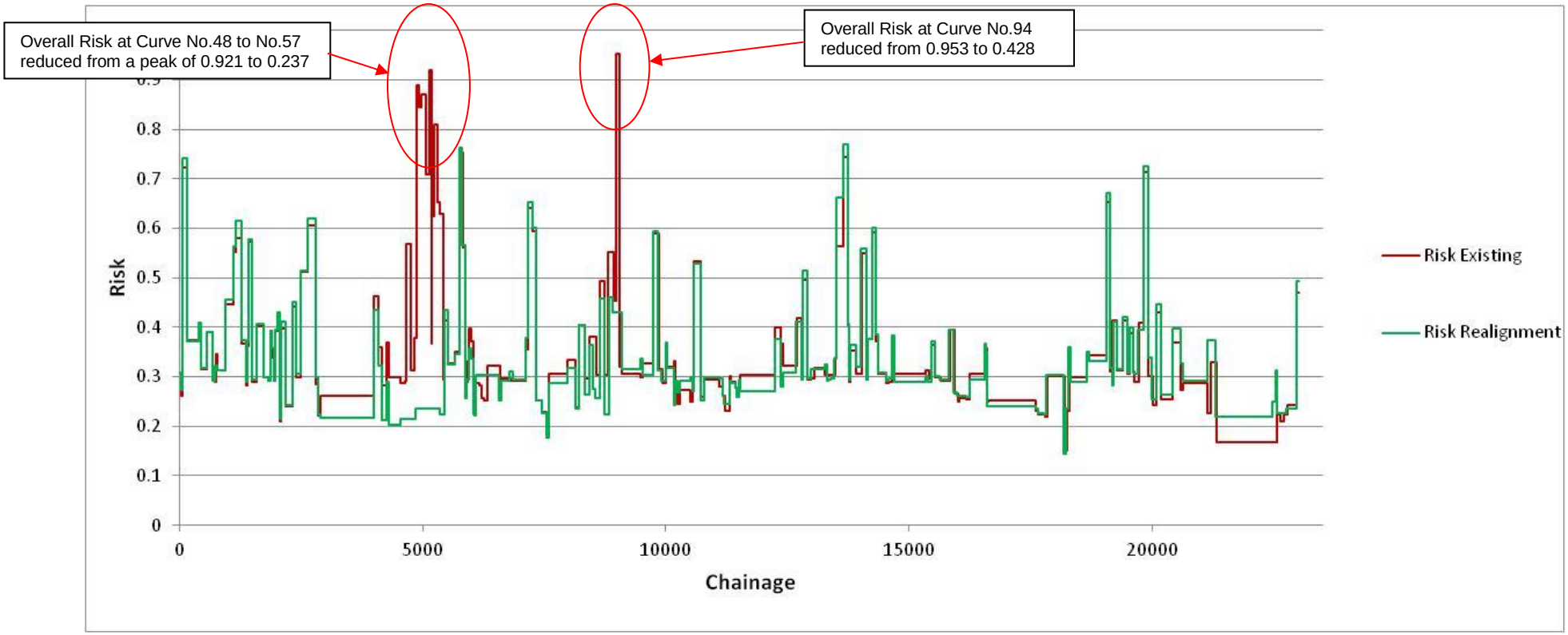


Figure 5.10 – N66 Comparison of Overall Risk Ratings

Speed Profile Comparison

The Speed Model shows an increase in Operating Speed from 67.1 km/h to 78.8 km/h at Curve No.94, and an increase from 28.4km/h to 82.9 km/h at Curve No.48 to No.57.

The speed profile shows that the Average Operating Speed on the entire length of the N66 increases slightly from 86.7km/h to 88.1 km/h.

The Operating Speed along the existing N66 ranges from a high of 98.3km/h to a low of 28.4km, equating to a speed variation along the route of 69.9km/h, giving a Consistency Rating of Very Poor. With the two local realignments the Operating speed would vary between 98.3 km/h and 71.8 km/h, giving a greatly reduced speed variation of 26.5 km, equating to a Consistency Rating of Poor, a considerable improvement from the existing rating of Very Poor. This indicates that there are several other relatively poor sections of the route that could be improved to further narrow the speed variation to achieve a Consistency Rating of Fair or Good.

Figure 5.11 shows a comparison of the existing Operating Speed Profile and the indicative realignment Operating Speed Profile. This shows that the Operating Speed has increased significantly where indicative realignments have been modelled making it more consistent with adjacent sections.

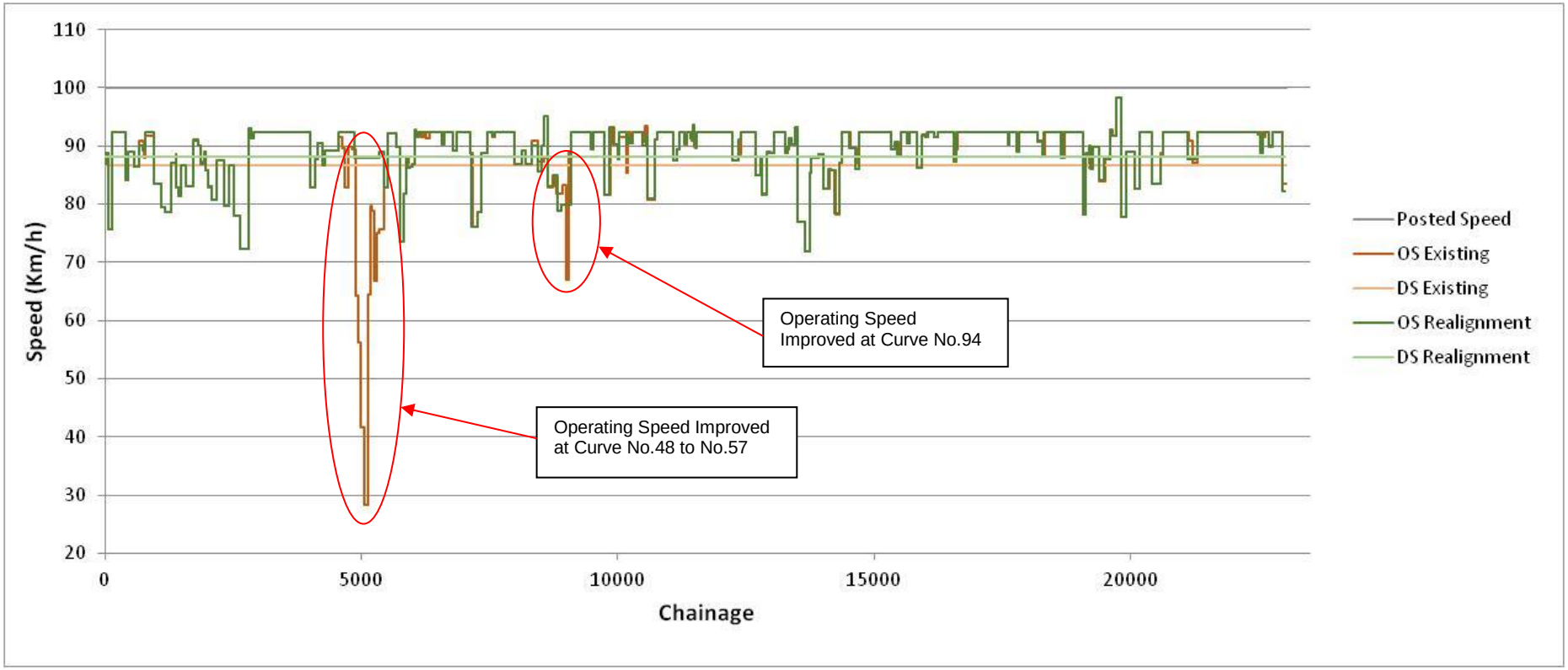


Figure 5.11 – Operating Speed Impacts of Realignment on the N66

5.1.8 N66 Case Study Conclusions

1) The (former) N66 was selected for this case study as it contained four of the pilot sites in this study. It is a good example of a typical legacy road.

2) The existing alignment contains some curves with radii as low as 40m, and 37% of the curves are lower than desirable minimum for 100 km/h.

3) The Overall Risk Rating of the existing N66 rages from 0.151 to 0.953.

4) The Risk Model identifies two areas with particularly high Risk Rating, which coincide with the two Collision clusters.

5) Two potential realignment options were developed for the two selected sites with the highest risk ratings.

6) The Risk Model indicates that the optimal solutions are a reduced standard realignment (400m length) at one location, and a desirable minimum standard realignment (800m length) at the other.

7) The Overall Risk Rating falls from 0.953 to 0.428 and from a range of 0.921 to 0.237 at the two locations.

8) With the indicative realignments the Operating Speeds increases from 67.1 km/h to 78.8 km/h, and from 28.4km/h to 82.9 km/h at the two locations.

9) The overall Average Operating Speed for the N66 increases from 86.7 km/h to 88.1 km/h.

10) Speed Variation along the N66 reduces from 69.9km/h to 26.5km/h.

11) The Consistency Rating increases from Very Poor to Poor, which understates the very large improvement achieved in absolute terms.

12) The improvement in Overall Risk Rating and Operating Speed is the consequence of two modest realignment schemes totalling 1.2 km (5.2%) of a 23 km long route.

13) An overall improvement strategy for the 23km length of this sample route could tackle the next highest risk sites with ratings >0.6, or a larger set >0.5, to optimise the overall risk performance along this route.

5.2 N71, County Cork, Case Study

5.2.1 N71 Case Study Route Details

The N71 is a 189km length of National Secondary single carriageway road between Cork City, Co. Cork and Killarney, Co. Kerry. The route passes primarily through rural countryside, generally following the south-western coastline. The route passes through the villages and towns of Innishannon, Bandon, Clonakilty, Lisavaird, Rosscarbery, Skibbereen, Bantry, Glengarriff, Kenmare and Killarney. The N71 route forms part of the Ring of Kerry.

The study route is the 6 km length of the N71 between Inishannon and Bandon where the road typically follows the line of the Bandon River and crosses the river in Inishannon. The extents of the N71 study route are indicated on Figure 5.12.

The character of the road layout on the N71 between Inishannon and Bandon is typically bendy. The Inishannon end of the section is bounded by the Bandon River on the north and by high ground (approximately 60m higher than the road) to the south resulting in a bendier alignment. At the Bandon end of the section, the terrain to the south of the road becomes flatter and the alignment becomes straighter.

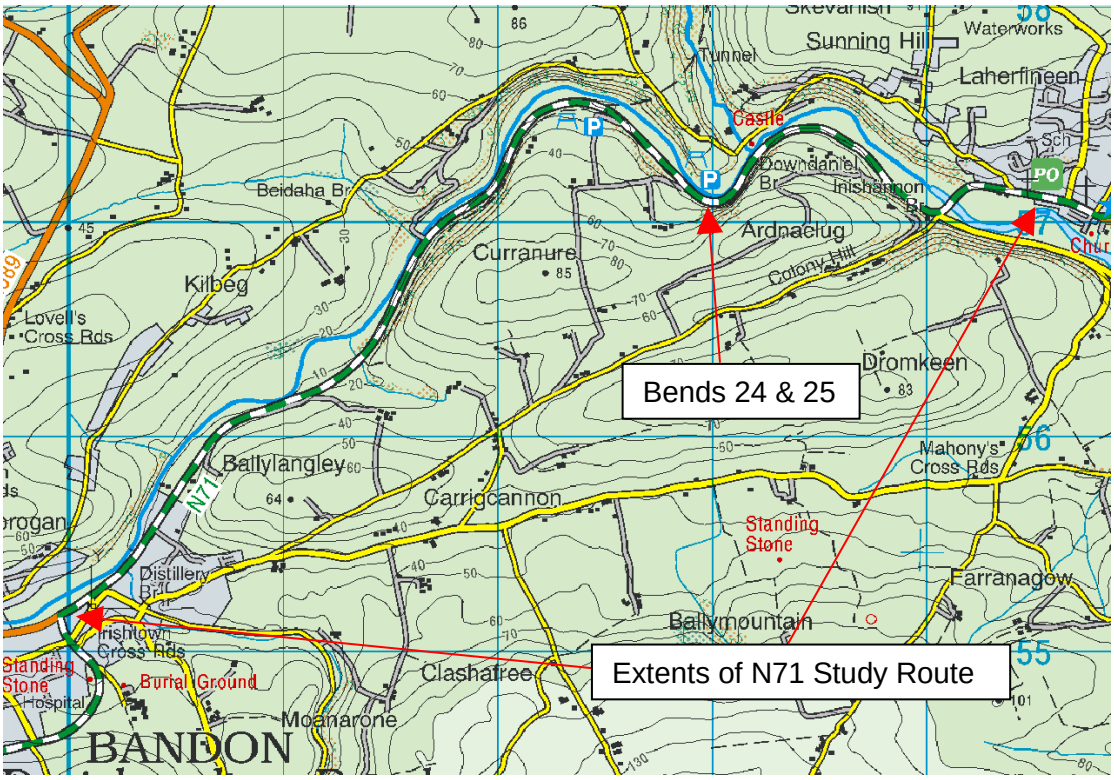


Figure 5.12 – N71 Case Study Route

5.2.2 Recorded Collision History

A review of the recorded collisions along the N71 indicates that the following collisions occurred:

1) Fatal Collisions – 1 No. west of Inishannon in 2011;
2) Serious Collisions – 1 No. approximately midway between Inishannon and Bandon;
3) Minor Collisions – 15 No. spread along the route with two groups of 6 No. collisions west of Inishannon;
4) Material Damage Collisions – 35 No. including 13 No. at the site of the 2011 fatal collision and groupings of collisions at the approaches to Inishannon and Bandon.

The collision data indicates that the 2011 fatal collision, 6 No. minor collisions and 13 No. material damage collisions occurred at the tightest bend on the section, a 96m Radius bend referenced as Bend No. 24 in this technical note and at a 190m Radius bend referenced as Bend No. 25. The locations of recorded collisions on the N71 are indicated on Figure 5.13.

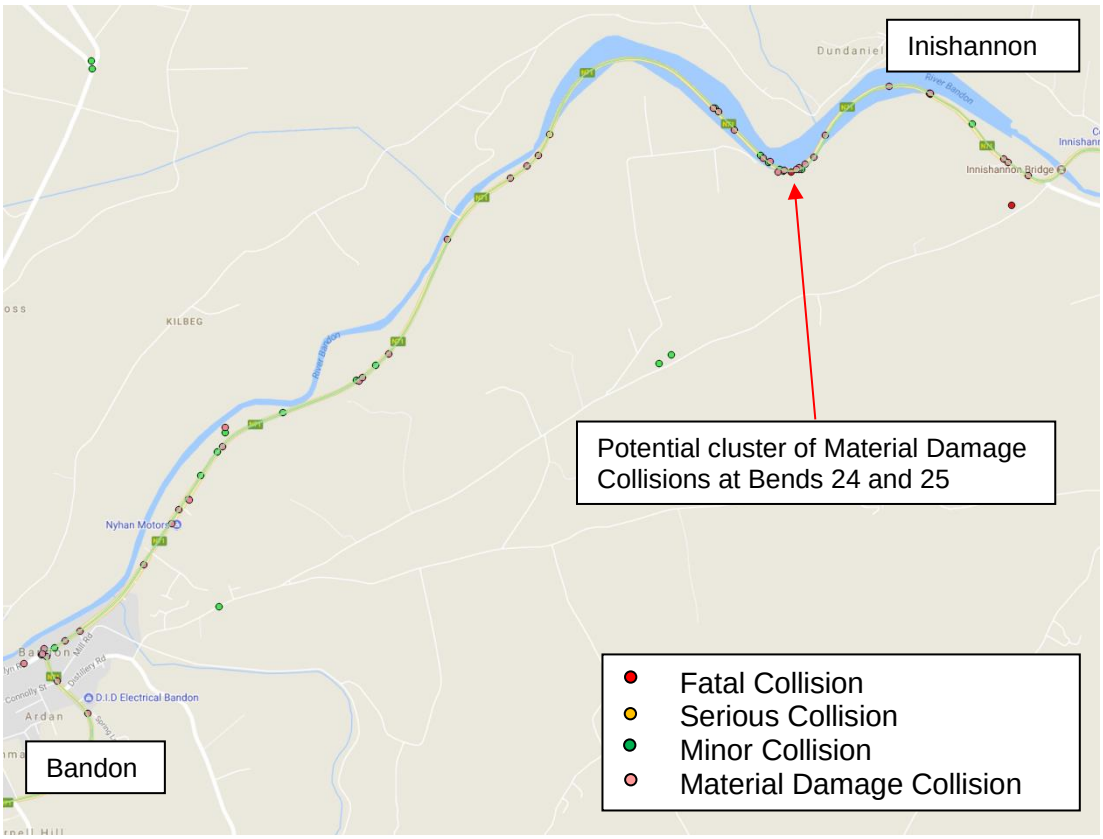


Figure 5.13 – N71 Collision History

5.2.3 Analysis of Existing N71 Route

Existing Alignment Derivation

The alignment derived for the N71 route between Inishannon and Bandon indicates the presence of 37 horizontal curves along its length, with curve radii ranging from 77m to 1,937m. A review of the posted speed limits on this section of the N71 indicated that parts of this derived alignment are located within the reduced 50 km/h speed limit zones for Inishannon and Bandon. The 7 No. bends on these sections are not included in the following analysis. The horizontal curves can be grouped into bands as per Table 5.11.

Table 5.11 – N71 Existing Horizontal Alignment Curve Radii

Curve Radius	DN-GE0-03031 Standard for 100km/h Design Speed (Table 1.3)	Number of Curves
<127m	Beyond Standard	1 (3.3%)
127m - 180m	Beyond Standard	3 (10.0%)
180m - 255m	Four Steps Below Desirable Minimum	3 (10.0%)
255m - 360m	Three Steps Below Desirable Minimum	5 (16.7%)
360m – 510m	Two Steps Below Desirable Minimum	5 (16.7%)
510m – 720m	One Steps Below Desirable Minimum	4 (13.3%)
>720m	Desirable Minimum	9 (30.0%)

The analysis of the N71 route alignment indicates that 12 out of the 30 horizontal curves (40%), are below requirements of Table 1.3 of TII Publication DN-GEO-03031 for a Design Speed of 100km/h. This does not take into account the super-elevation requirements of Table 1.3 of DN-GEO-03031 for horizontal curves.

The alignment derived for the N71 between Inishannon and Bandon indicates that the radius of Curve No. 24 is 97m which is below the minimum radius requirements of Table 1.3 of DN-GEO-03031 for a Design Speed of 100km/h. This is the tightest radius curve on this section of the N71.

Overall Risk Rating for Existing Route

The Overall Risk Ratings for each element of the existing N71 route range from a lowest Overall Risk Rating of 0.020 to a highest Overall Risk Rating of 0.711. Figure 5.14 shows the Overall Risk profile along the N71 route as determined by the Risk Model. Table 5.12 shows the top 15 Overall Risk Rated alignment elements for the route.

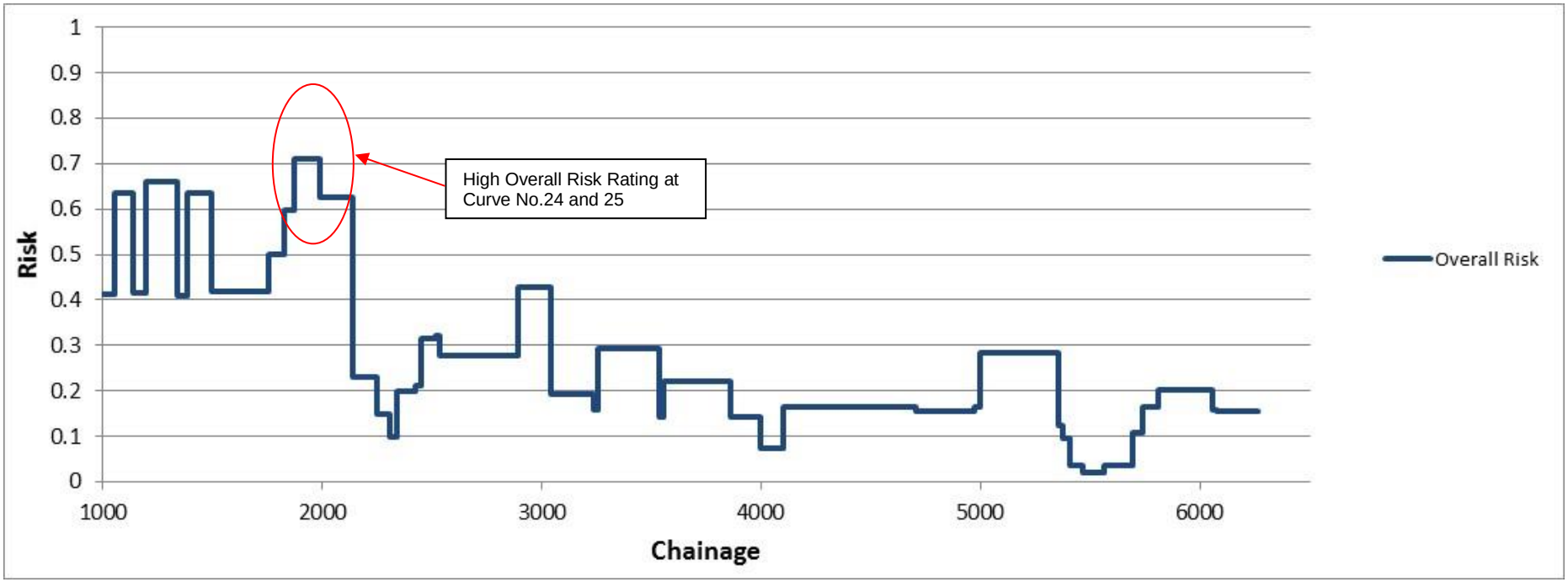


Figure 5.14 – N71 Existing Overall Risk Rating Profile

Table 5.12 – N71 Existing Top 15 Overall Risk Rated Elements

Risk Order	Alignment Element ID	Alignment Element	Start Chainage	End Chainage	Overall Risk
1	24	Bend	1872.000	1991.000	0.711
2	18	Bend	1200.000	1339.000	0.661
3	16	Bend	1055.000	1138.000	0.637
4	20	Bend	1389.000	1495.000	0.634
5	25	Bend	1991.000	2137.000	0.625
6	23	Bend	1825.000	1872.000	0.598
7	22	Bend	1756.000	1825.000	0.499
8	34	Bend	2893.000	3040.000	0.427
9	21	Bend	1495.000	1756.000	0.419
10	17	Bend	1138.000	1200.000	0.415
11	15	Bend	956.000	1055.000	0.414
12	19	Bend	1339.000	1389.000	0.410
13	32	Tangent	2514.000	2536.500	0.322
14	31	Bend	2452.500	2514.000	0.315
15	37	Bend	3256.500	3536.000	0.293

Table 5.12 indicates that the highest Overall Risk of 0.711 occurs at a pair of back to back curves (Curves No.24 & 25) between Ch.1+872 and Ch.2+137, highlighted in yellow.

Operating Speed Profile for Existing N71 Route

The Operating Speed Profile indicates that the operating speed along the existing N66 route ranges from a high of 93 km/h to a low of 66 km/h. This equates to a speed variation along the route of 27km/h. In accordance with the *Geometric Design Consistency Model* developed by Lamm et al. (1999), a speed range in excess of 20 km/h indicates a Consistency Rating of Poor. The existing N71 route falls into this band. Figure 5.15 shows a Design Speed (Average Operating Speed) of 86km/h.

Figure 5.15 indicates that the location where the greatest speed variation occurs coincides with the location where the Risk Model determined the highest risk occurs.

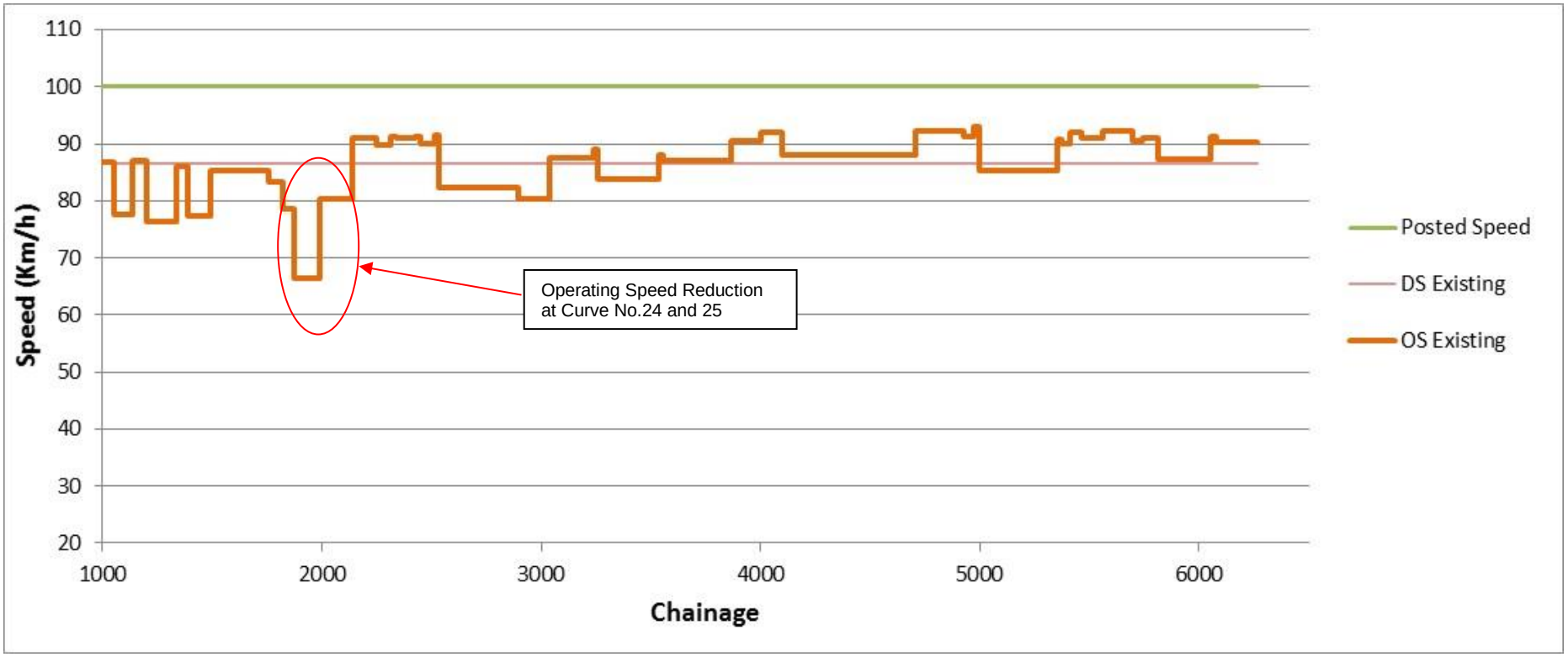


Figure 5.15 – N71 Existing Operating Speed Profile

Risk Analysis of Existing Curve No. 24 and No.25

The highest Overall Risk Ratings of 0.711 and 0.625 were observed between Ch.1+872 and Ch.2+137, highlighted in yellow in Table 5.12 and Table 5.13. This pair of curves coincides with an apparent cluster of collisions. Table 5.13 shows the alignment elements in the vicinity of the pair of bends. The Overall Risk Rating associated with Curve No.24 to No.25 are higher than the Overall Risk Rating for the adjacent alignment elements, particularly for the alignment elements on the eastbound approaches to the bends (Curves 26 to 30).

Table 5.13 – Existing N71 Overall Risk Ratings for Element No. 15 to No. 35

Alignment Element ID	Alignment Element	Start Chainage	End Chainage	Curve Radius (m)	Overall Risk Rating	Risk Order
15	Bend	956.000	1055.000	466	0.414	16
16	Bend	1055.000	1138.000	158	0.637	6
17	Bend	1138.000	1200.000	507	0.415	15
18	Bend	1200.000	1339.000	139	0.661	5
19	Bend	1339.000	1389.000	403	0.410	17
20	Bend	1389.000	1495.000	159	0.634	7
21	Bend	1495.000	1756.000	365	0.419	14
22	Bend	1756.000	1825.000	267	0.499	11
23	Bend	1825.000	1872.000	197	0.598	9
24	Bend	1872.000	1991.000	97	0.711	4
25	Bend	1991.000	2137.000	190	0.625	8
26	Tangent	2137.000	2252.000		0.230	29
27	Bend	2252.000	2311.000	806	0.149	46
28	Tangent	2311.000	2339.500		0.099	51
29	Bend	2339.500	2423.000	1550	0.198	33
30	Tangent	2423.000	2452.500		0.212	31
31	Bend	2452.500	2514.000	1289	0.315	23
32	Tangent	2514.000	2536.500		0.322	22
33	Bend	2536.500	2893.000	297	0.277	27
34	Bend	2893.000	3040.000	217	0.427	13
35	Bend	3040.000	3237.000	579	0.194	34

5.2.4 Analysis of Indicative Realignments on N71 Route

An indicative realignment scheme was developed for the highest collision risk, Curves No.24 and No. 25. This indicative realignment scheme comprised an alignment with horizontal curvature relaxations of 4 steps below Desirable Minimum.

Due to the very difficult nature of the terrain alongside the River Bandon in the vicinity of Curve Nos. 24 and 25, realignment options are severely limited. An alignment based on the Desirable Minimum horizontal curvature was considered; however, a

realignment based on the desirable minimum standards would have resulted in a much larger realignment scheme than would typically be considered as a minor improvement scheme. This realignment was therefore not progressed as part of this case study.

5.2.5 Analysis of Curve No.24 and No.25 Realignment

Curve No.24 and No.25 Option 1

An indicative realignment scheme was developed at this location. The modest realignment involved the replacement of Curve Nos. 22 to 26 with a pair of 180m radius curves. A curve radius of 180m equates to a 4 step below Desirable Minimum curve for a Design Speed of 100km/h and would therefore be below the standard required by with DN-GEO-03031. The realignment of the N71 using a reduced standard alignment at this location would require the realignment of 0.5 km of road and would likely not require the acquisition of any properties but would require the construction of two significant bridges across the Bandon River. The indicative Option 1 realignment at Curve No.24 and No.25 is shown on Figure 5.16.

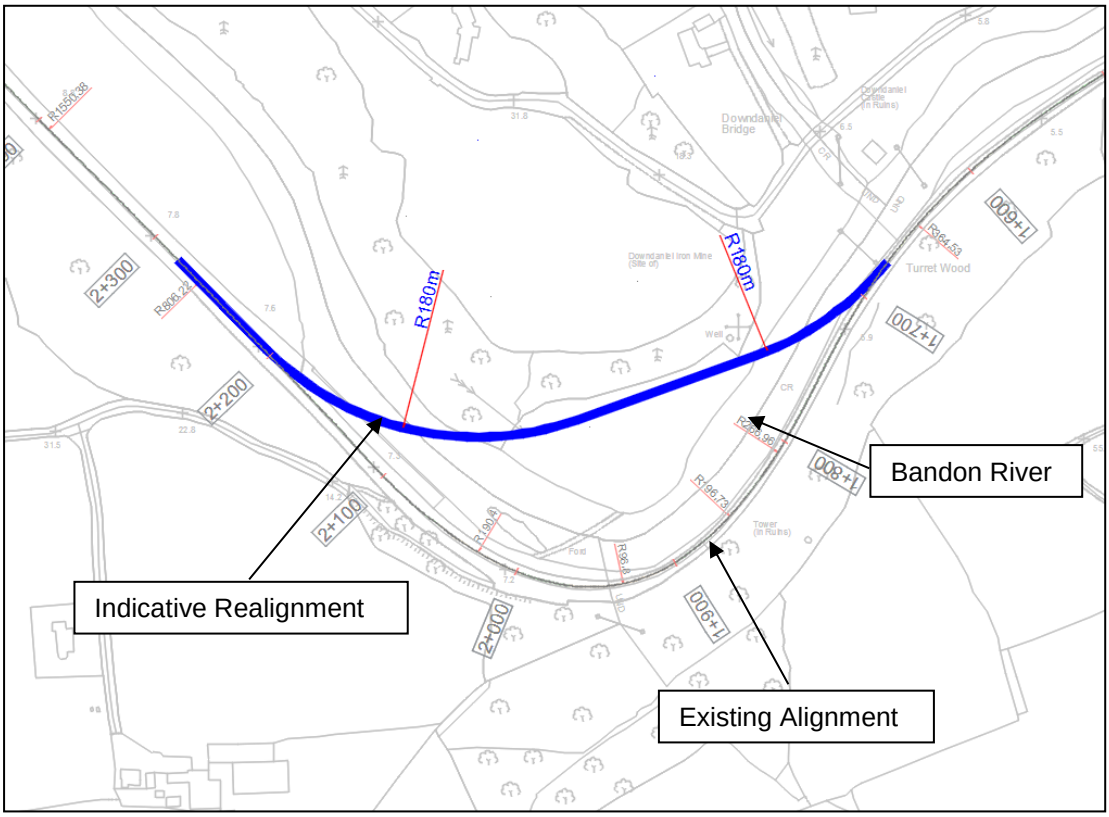


Figure 5.16 – N71 Curve 25 and 26 Option 1 Alignment

The Risk Model was re-run using this realignment option to demonstrate the impact on the Overall Risk Rating of Curve Nos.24 and 25, and the adjacent curves. The results of this analysis are shown in Table 5.14.

Table 5.14 – N71 Option 1 Alignment Risk Ratings

Alignment Element ID	Alignment Element Type	Curve Radius/m	Overall Risk Rating	Risk Order
15	Bend	466	0.422	13
16	Bend	158	0.638	6
17	Bend	507	0.433	11
18	Bend	139	0.663	4
19	Bend	403	0.411	15
20	Bend	159	0.640	5
21	Bend	365	0.421	14
22	Bend	180	0.457	10
23	Tangent		0.148	43
24-27	Bend	180	0.609	7
28	Tangent		0.190	31
29	Bend	7100	0.162	35
30	Tangent		0.146	44
31	Bend	1289.141	0.186	34
32	Tangent		0.193	30
33	Bend	297.108	0.278	24
34	Bend	216.985	0.428	12
35	Bend	579.495	0.235	25

Curve No. 24 and No.25 Indicative Realignment Comparison

Table 5.15 – Comparison of N71 Alignment Risk Ratings for Options

Alignment Element ID	Alignment Element Type	Existing Alignment		Option 1 Alignment	
		Overall Risk	Risk Order	Overall Risk	Risk Order
15	Bend	0.414	16	0.422	13
16	Bend	0.637	6	0.638	6
17	Bend	0.415	15	0.433	11
18	Bend	0.661	5	0.663	4
19	Bend	0.410	17	0.411	15
20	Bend	0.634	7	0.640	5
21	Bend	0.419	14	0.421	14
22	Bend	0.499	11	0.457	10
23	Bend	0.598	9		
24	Bend	0.711	4	0.609	7
25	Bend	0.625	8		
26	Tangent	0.230	29		
27	Bend	0.149	46		
28	Tangent	0.099	51	0.190	31
29	Bend	0.198	33	0.162	35
30	Tangent	0.212	31	0.146	44
31	Bend	0.315	23	0.186	34
32	Tangent	0.322	22	0.193	30
33	Bend	0.277	27	0.278	24
34	Bend	0.427	13	0.428	12
35	Bend	0.194	34	0.235	25

Table 5.15 shows a comparison of the Overall Risk Rating at Curve No.22 to No.27. for the existing alignment and the realignment option. The Overall Risk Rating at Curve No.22 to No.27 reduces from a high of 0.711 for the existing alignment and to a high of 0.609 for the realignment. The Overall Risk Ratings for the realignment at Curve No.22 to No.27 remains higher than with the Overall Risk Rankings for the adjacent elements indicating that the indicative realignment will reduce the Overall Risk Ranking at the location, however, the risk will not be reduced to a level that is consistent with the remainder of the route. The nature of the terrain and the presence and alignment of the Bandon River in the vicinity of Curve Nos. 24 and 25, realignment options are limited. In order to reduce the risks further, a much larger realignment scheme would need to be considered. This scale of realignment would be beyond the scale of realignment was that is considered as part of this project.

5.2.6 N71 Alignment Assessment Findings

Overall Risk Reduction

On the basis of the assessment of the existing N71 alignment and the indicative realignment developed at the highest risk location, the Risk Model was re-run using the realignment at Curve No. 25 and No. 26. This realignment equates to a total realignment length of 0.5 km on an overall route length of 6km. This re-analysis showed a drop in the peak Overall Risk Rating from 0.711 to 0.609.

Figure 5.17 shows a comparison of the existing Overall Risk Profile and the indicative realignment Overall Risk Profile. This shows that the Overall Risk Profile has reduced where the indicative realignment has been modelled.

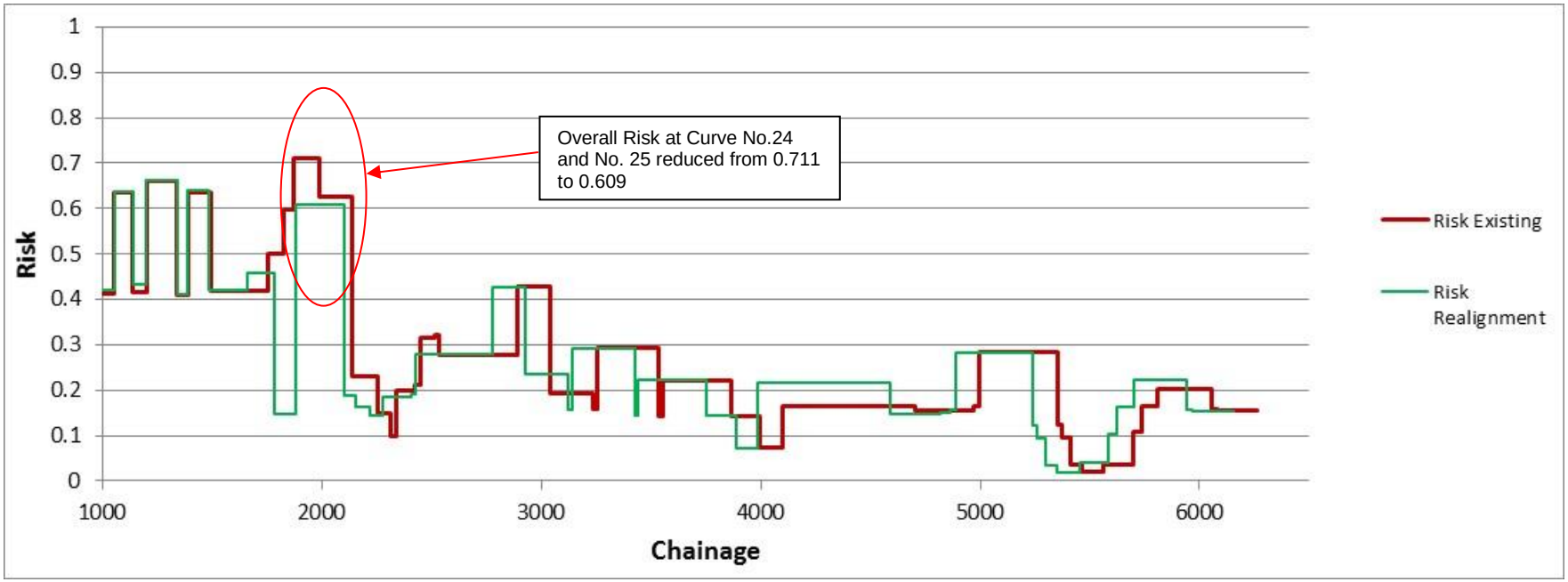


Figure 5.17 – N71 Comparison of Overall Risk Ratings

Speed Profile Comparison

The Speed Model shows an increase in Operating Speed from 66 km/h to 76 km/h at Curve No. 24 and No.25.

The speed profile shows that the Average Operating Speed on the entire length of the N71 increases slightly from 86 km/h to 87 km/h.

The Operating Speed along the existing N71 ranges from a high of 93km/h to a low of 66km, equating to a speed variation along the route of 27km/h, giving Consistency Rating of Poor. Based on the indicative realignment Operating Speed, the Operating Speed would vary between 93 km/h and 76 km/h, a speed variation of 17 km/h, equating to a Consistency Rating of Fair, an improvement from the existing rating of Poor.

Figure 5.18 shows a comparison of the existing Operating Speed Profile and the indicative realignment Operating Speed Profile. This shows that the Operating Speed has increased where indicative realignment has been modelled making it more consistent with adjacent sections.

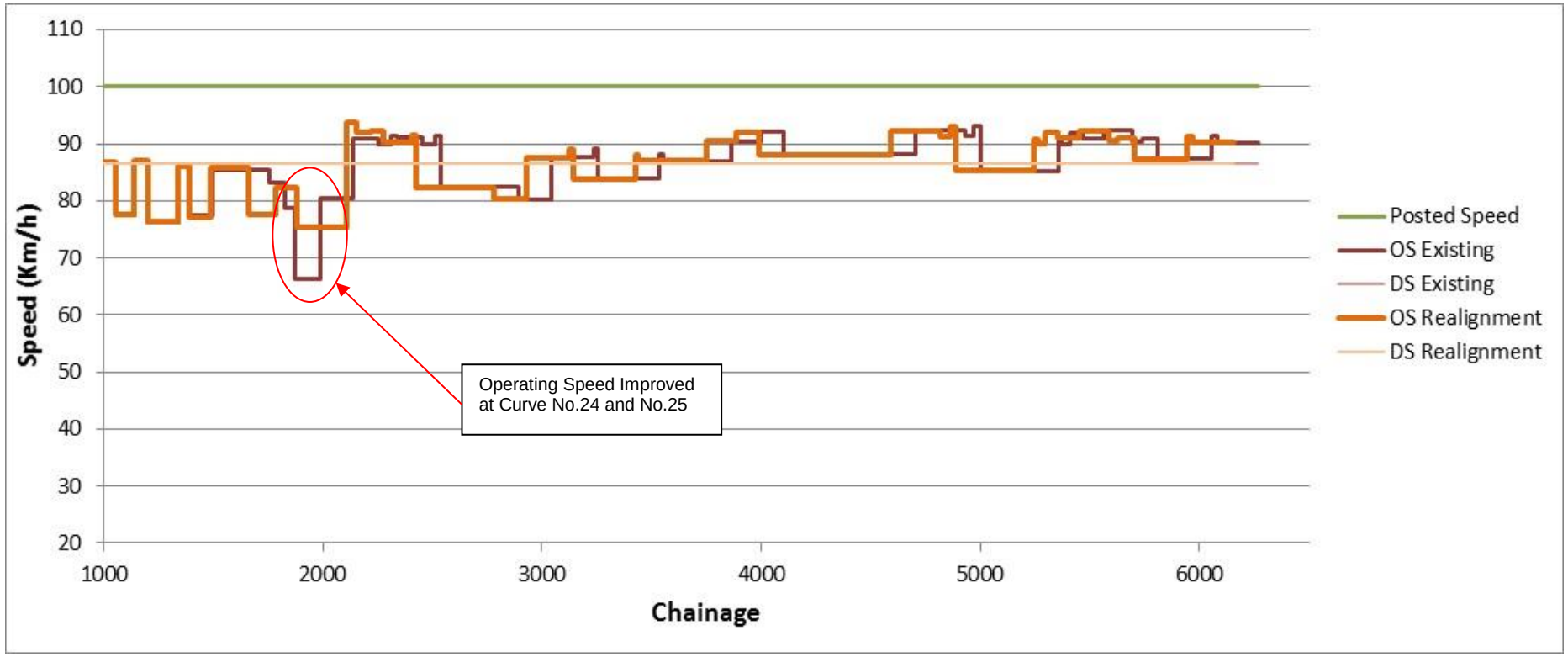


Figure 5.18 – N71 Comparison of Operating Speeds

5.2.7 N71 Case Study Conclusions

1) The collision data indicates the presence of clusters of Collisions at Bend No.24 and No.25

2) The existing alignment contains horizontal curves with radii as low as 77m, and 40% of the curves within the 100km/h section of the road were found to be below the standard required for a Design Speed of 100km/h.

3) The Overall Risk Rating of the existing N71 rages from 0.020 to 0.714.

4) The Risk Model identifies that the highest Overall Risk Rating alignment element is a 97m radius Curve, Curve No.24 which coincides with a Material Damage Collision cluster.

5) A 500m long indicative realignment, comprising bends with radii of 4-steps below the Desirable Minimum was developed at the highest Over Risk Rating location.

6) The Risk Model indicates the Risk Rating falling from 0.711 to 0.609.

7) Operating Speed increases from 66 km/h to 76 km/h at Curve No.24.

8) The Average Operating Speed for the N71 increases slightly from 86km/h to 87 km/h.

9) Speed Variation falls from 27 km/h to 17 km/h.

10) The Consistency Rating increases from Poor to Fair.

11) The improvement in Overall Risk Rating and Operating Speed is the consequence of one modest realignment scheme totalling 0.5km (8.33%) of a 6 km long route.

12) An overall improvement strategy for the 6km length of this sample route could tackle the next highest risk sites with ratings >0.5, to optimise the overall risk performance along this route.

5.3 N76, County Tipperary, Case Study

5.3.1 N76 Case Study Route Details

The N76 is a 44km length of National Secondary single carriageway road between Clonmel, Co. Tipperary and Kilkenny, Co. Kilkenny. The route passes primarily through rural rolling countryside. The route passes to the west of the town of Callan, Co. Kilkenny and through the villages of Ninemilehouse, Mullennaglogh and Grange Mockler, Co. Tipperary. The N76 terminates at a roundabout on the N10 Kilkenny Ring Road and at the Killheffernan Roundabout on the N24, east of Clonmel. The posted speed limit on the entire length of the case study route is 100 km/h.

The case study route is the 7 km length of the N76 between the Killheffernan Roundabout and Kilcash Cross. The extents of the N76 study route are indicated on Figure 5.19.

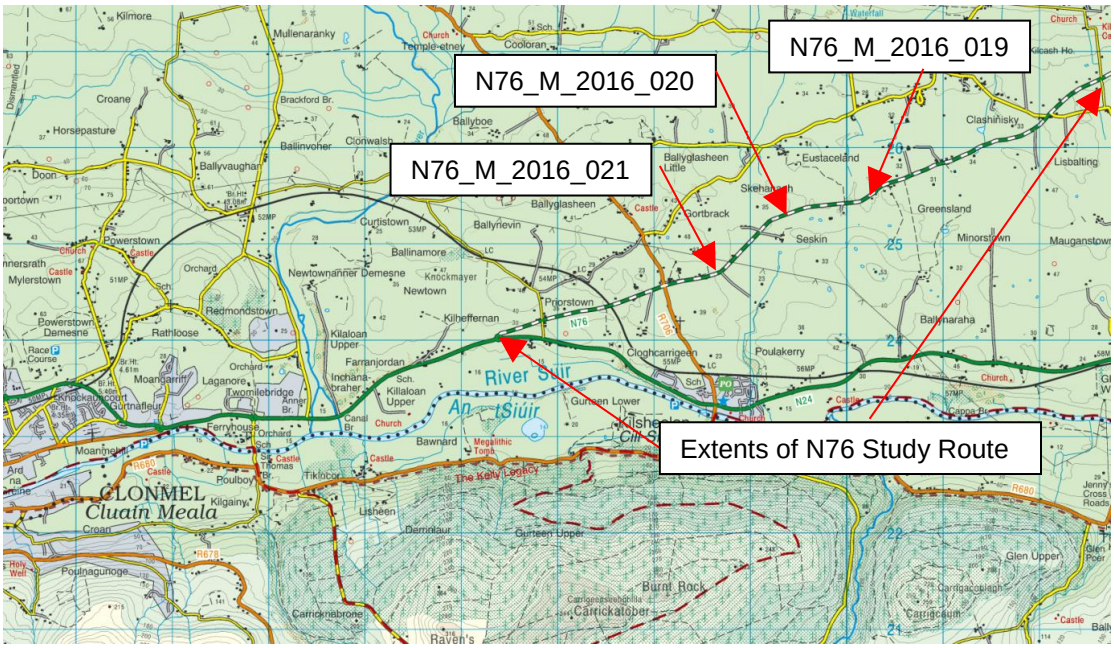


Figure 5.19 – N76 Study Route

The character of the road layout on the N76 at Seskin is typically straight with intermittent short curves. The route traverses rolling countryside, resulting in a rolling vertical alignment.

Resurfacing works were carried out on this section of the N76 in 2012 with pavement surface retexturing works carried out in late 2016 / early 2017 due to concerns over the pavement skid resistance provided by the pavement.

Traffic speed surveys were carried out by Tipperary County Council in March 2018 over a period of 7 days. These speed surveys indicate that the average vehicle speed on this section of the N76 is 72 km/h with an 85th percentile speed of below 80 km/h. It should be noted that at the time of these speed surveys, variable message signs

(VMS) displaying a warning to slow down and a suggested speed of 60 km/h were in place on the route, which may have affected driver speeds. The speed survey results provided by Tipperary County Council also indicate that that there was snow on one of the survey days, which is also likely to have had an impact on vehicle speeds. The speed survey results may therefore underestimate the free-flow traffic speeds on the N76 and should be treated with caution. The speed survey indicates that the highest recorded speed was 139 km/h.

5.3.2 Recorded Collision History

A review of the recorded collisions along the N76 indicates that the following collisions occurred:

1) Fatal Collisions – 1 No. at site N76_M_2016_21 in 2008;
2) Serious Collisions – 2 No. at site N76_M_2016_20 (2006) and N76_M_2016_21 (1997);
3) Minor Collisions – 3 No. at sites N76_M_2016_19 (2004), N76_M_2016_20 (2001) and N76_M_2016_21 (1996);
4) Material Damage Collisions – 12 No. including 9 No. at site N76_M_2016_19, 2 No. at site N76_M_2016_20 and 1 No. at site N76_M_2016_19;
5) Unclassified Collisions – 4 No. including 2 No. at site N76_M_2016_19 and 1 No. at sites N76_M_2016_20 and N76_M_2016_21.

The information received in relation to the 4 recent unclassified collisions recorded by TCC indicates that the collisions were all single vehicle collisions that occurred when the road surface was wet.

The locations of collisions recorded by the RSA and TII on the N76 are indicated on Figure 5.20. This figure does not include the locations of the 4 unclassified collisions recorded by TCC.

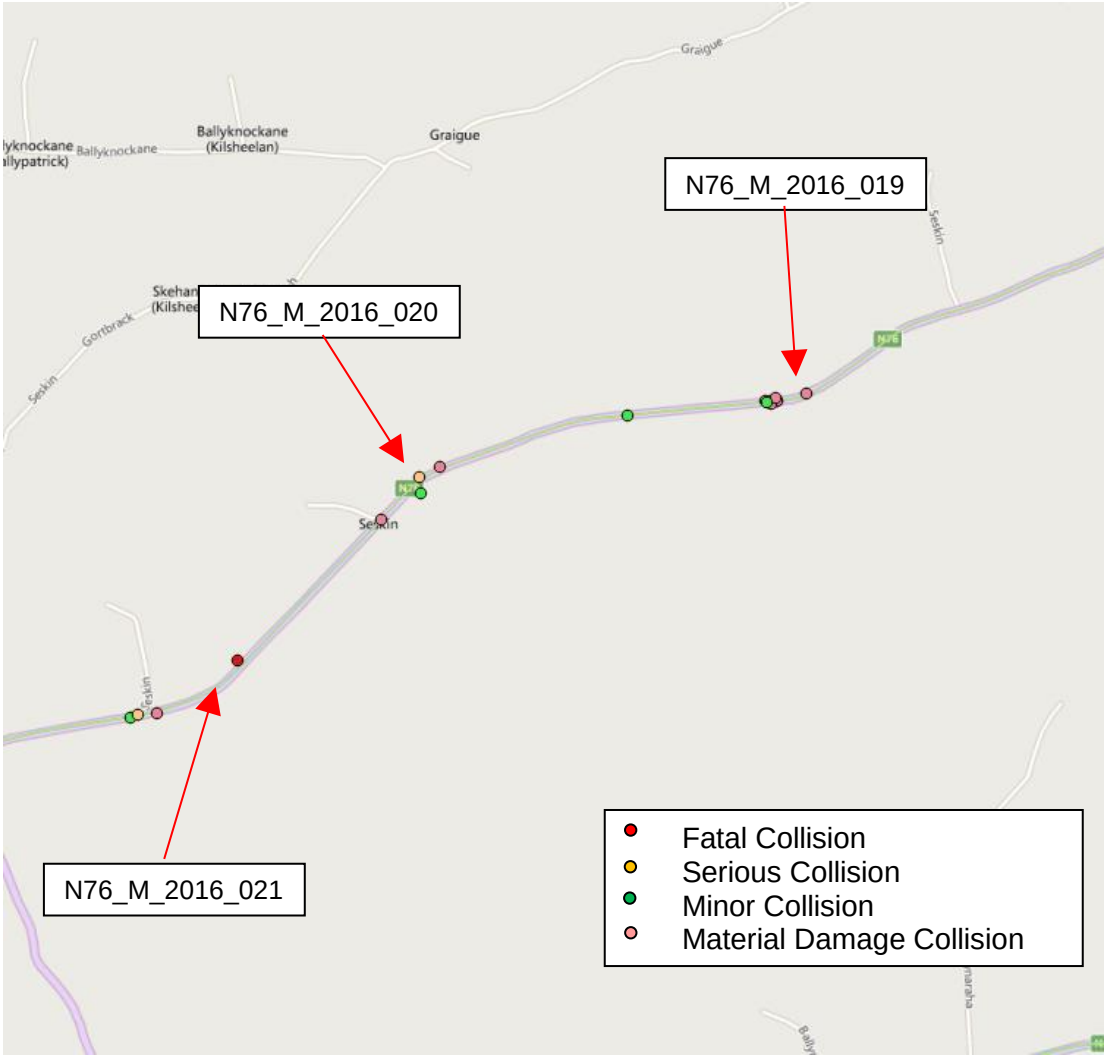


Figure 5.20 – N76 Collision History

5.3.3 Analysis of Existing N76 Route

Existing Alignment Derivation

The alignment derived for the N76 route between the Killheffernan Roundabout and Kilcash Cross indicates the presence of 59 horizontal curves along its length, with curve radii ranging from 43m to 8,800m. The alignment analysis indicates the presence of the horizontal curves that can be grouped into bands as per Table 5.16.

Table 5.16 – N76 Existing Horizontal Alignment Curve Radii

Curve Radius	DN-GE0-03031 Standard for 100km/h Design Speed (Table 1.3)	Number of Curves
<127m	Beyond Standard	1 (1.7%)
127m - 180m	Beyond Standard	2 (3.4%)
180m - 255m	Four Steps Below Desirable Minimum	5 (8.5%)
255m - 360m	Three Steps Below Desirable Minimum	6 (10.2%)
360m – 510m	Two Steps Below Desirable Minimum	8 (13.6%)
510m – 720m	One Steps Below Desirable Minimum	7 (11.9%)
>720m	Desirable Minimum	30 (50.8%)

The analysis of the N76 route alignment indicates that 14 out of the 59 horizontal curves (23.7%), are of a radius that is below the requirements of Table 1.3 of TII Publication DN-GEO-03031 for a Design Speed of 100km/h. This rate does not take into account the super-elevation requirements of Table 1.3 of DN-GEO-03031 for horizontal curves.

The alignment derived for the N76 at Seskin indicates that the radii of bends N76_M_2016_019, 020 and 021 are 163m, 196m and 214m respectively, all below the minimum radius requirements of Table 1.3 of DN-GEO-03031 for a Design Speed of 100km/h. These are the three tightest radii curves on this section of the N76 with the exception of two curves on the immediate approach to the Killheffernan Roundabout, which can be considered to form part of the junction.

The derived alignment indicates that the horizontal curve at Bend 57 is a 163m radius curve. This radius compares to a Desirable Minimum radius of 720m in accordance with DN-GEO-03031, Table 1.3. Paragraph 3.5 of DN-GEO-03031 allows for a 3-step below Desirable Minimum to a radius of 255m with 7% super-elevation. The existing radius of 163m effectively equates to 5-steps below Desirable Minimum and is therefore beyond the scope of DN-GEO-03031.

In addition to the routine SCRIM surveys carried out by TII, Road Profiling surveys of the network are also given to determine the ride quality on roads within the network. This Road Profiling data includes information on the cross-fall / super-elevation along routes. The data for the N76 route indicates that Bend 57 is super-elevated to 2% to 2.5% approximately. Table 1.3 of DN-GEO-03031 indicates that for a 100km/h Design

Speed, the Desirable Minimum radius curve of 720m requires super-elevation of 5% and a One Step below Desirable Minimum of 510m requires super-elevation of 7%. As stated above, a curve radius of 163m effectively equates to 5-steps below Desirable Minimum, which would include the provision of 7% super-elevation. The level of super-elevation provided at Bend 57 is therefore also beyond the scope of DN-GEO-03031.

The vertical alignment derived for this section of the N76 indicates the presence of a crest curve between Ch.6+137 and Ch.6+220, approximately 100m east of bend 57. This is consistent with the location of a crest curve to the east of site N76_M_2016_019 observed on site. The derived vertical alignment indicates that this crest curve has a k-value of 16 and results in a stopping sight distance of 84m for westbound traffic. This compares to a Desirable Minimum crest k-value of 100 and Desirable Minimum stopping sight distance of 215m for a Design Speed of 100km/h in accordance with DN-GEO-03031, Table 1.3. The existing k-value and Stopping Site Distance at this location are effectively 3-steps below Desirable minimum and therefore below the standard required by DN-GEO-03031 for a Design Speed of 100km/h.

It is noted that the 2 collisions recorded by TCC at this location involved westbound vehicles, who would have encountered this crest curve in advance of the 163m horizontal curve. Observations made on site suggested that this crest curve obscures the visibility of westbound drivers to the curve ahead, including the warning chevron signs around the bend, and may lead drivers to misjudge the presence or severity of the curve.

It should also be noted that the application of combinations of relaxation are restricted in accordance with the requirements on DN-GEO-03031, limiting the number of steps below Desirable Minimum for horizontal curvature, vertical curvature and stopping sight distance.

Overall Risk Rating for Existing Route

The Overall Risk Ratings computed by the model for each element of the existing N76 route range from a lowest Overall Risk Rating of 0.144 to a highest Overall Risk Rating of 0.999. It should be noted that the two highest risk bends are located on the immediate approach to the Killheffernan Roundabout and can be considered to be part of the junction. These two curves are therefore not considered in the following assessment. Figure 5.21 shows the Overall Risk profile along the N76 route as determined by the Risk Model. Curve Nos. 57, 66 and 75, indicated on Figure 5.21, correspond to bends N76_M_2016_019, 020 and 021 respectively. Table 5.17 shows the top 15 Overall Risk Rated alignment elements for the route.

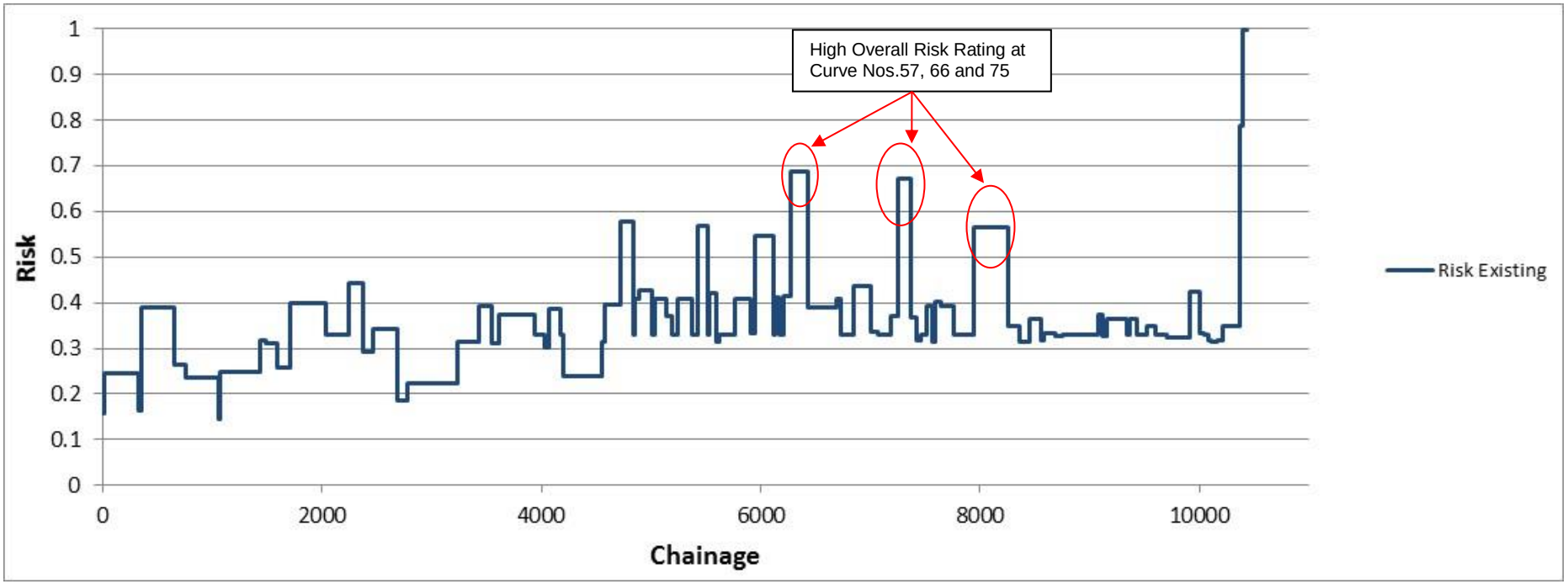


Figure 5.21 – N76 Existing Overall Risk Rating Profile

Table 5.17 – N76 Existing Top 15 Overall Risk Rated Elements

Risk Order	Alignment Element ID	Alignment Element Type	Start Chainage	End Chainage	Overall Risk
1	105	Bend	10393.034	10429.845	1.000
2	104	Bend	10373.032	10393.034	0.787
3	57	Bend	6269.023	6434.025	0.688
4	66	Bend	7251.026	7370.028	0.671
5	34	Bend	4718.017	4836.018	0.579
6	44	Bend	5421.520	5520.021	0.568
7	75	Bend	7940.528	8256.031	0.565
8	52	Bend	5948.521	6117.023	0.546
9	15	Bend	2251.011	2382.012	0.444
10	61	Bend	6842.025	6998.026	0.438
11	37	Bend	4898.018	5009.019	0.426
12	97	Bend	9917.532	9999.032	0.423
13	46	Bend	5537.521	5602.021	0.422
14	56	Bend	6210.023	6269.023	0.415
15	54	Bend	6132.023	6168.023	0.411

Table 5.17 indicates that the highest Overall Risks of 1.000 and 0.787 occur at two bends on the immediate approach to the Killheffernan Roundabout and can be considered to be part of the junction. These two curves are therefore not considered in the following assessment.

Table 5.17 indicates that the three curves identified by Tipperary County Council as being of particular concern, are amongst the highest risk ranked curves on this section of the N76. The three bends are highlighted in yellow in Table 5.17. As bend N76_M_2016_019 (bend 57) has the highest Overall Risk, this assessment focuses on this bend.

Operating Speed Profile for Existing Route

The Operating Speed Profile shown in Figure 5.22 indicates that the operating speed along the existing N76 route ranges from a high of 96 km/h to a low of 79 km/h, excluding the bends on the immediate approach to the Killheffernan Roundabout. This equates to a speed variation along the route of 17 km/h. In accordance with the *Geometric Design Consistency Model* developed by Lamm et al. (1999), a speed range of 10 km/h to 20 km/h indicates a Consistency Rating of Fair. The operating speed profile on the N76 falls into this band.

Figure 5.22 shows an Average Operating Speed of 89 km/h which compares with the measured 85th percentile vehicle speed of 80 km/h measured by TCC in March 2018 and the posted speed limit of 100 km/h.

Figure 5.22 indicates that the three locations where the greatest speed reduction occur coincide with the three locations identified by Tipperary County Council as being of particular concern and where the risk model determined that the highest risks occur.

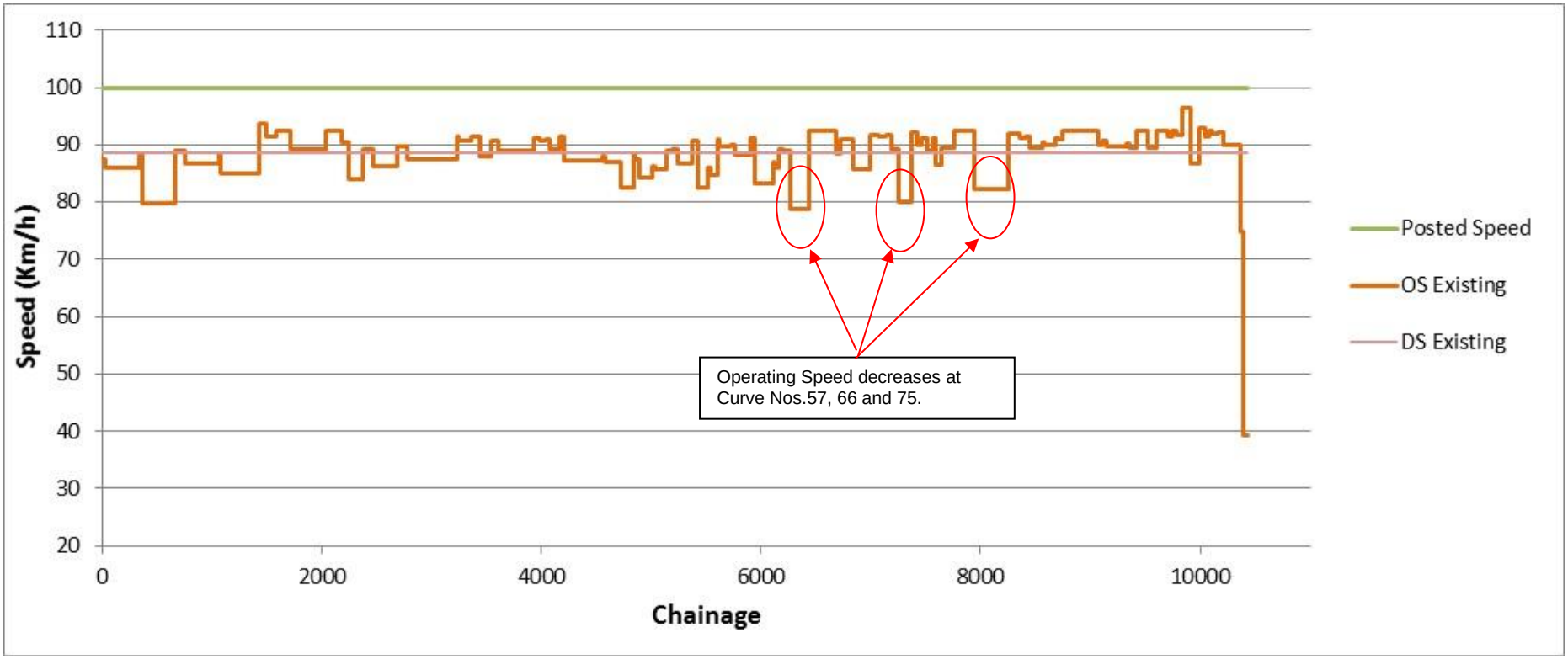


Figure 5.22 – N76 Existing Operating Speed Profile

5.3.4 Analysis of Existing Curve No. 57

The highest Overall Risk Rating of 0.688 was observed at alignment element Curve No. 57, a single 163m radius curve located between Ch.6+269 and Ch.6+434, highlighted in yellow in Table 5.18. This curve coincides with potential cluster of Collisions indicated by the recorded collision data at site N76_M_2016_019. Table 5.18 shows the alignment elements on either side of Curve No.57. Table 5.18 indicates that the Overall Risk Rating associated with Curve 57 is significantly higher than the Overall Risk Rating for the adjacent alignment elements.

Table 5.18 – Existing N76 Overall Risk Ratings for Element No. 52 to No. 62

Alignment Element ID	Alignment Element Type	Start Chainage	End Chainage	Curve Radius (m)	Overall Risk Rating	Risk Order
52	Bend	5948.521	6117.023	261	0.546	8
53	Tangent	6117.023	6132.023		0.330	60
54	Bend	6132.023	6168.023	433	0.411	15
55	Tangent	6168.023	6210.023		0.331	53
56	Bend	6210.023	6269.023	786	0.415	14
57	Bend	6269.023	6434.025	163	0.688	3
58	Tangent	6434.025	6686.025		0.390	28
59	Bend	6686.025	6729.025	590	0.409	17
60	Tangent	6729.025	6842.025		0.331	46
61	Bend	6842.025	6998.026	355	0.438	10
62	Tangent	6998.026	7074.526		0.337	42

The Overall Risk Ranking can be further broken down into the constituent risks Q_{CIi}, Q_{CIIi}, Q_{CIIIi}, Q_{SSDi}, Q_{CRRi}, Q_{VRRi} and Q_{wl} for each direction, as defined below:

Q_{CIi} Design Speed Variation

Q_{CIIi} Operating Speed Variation

Q_{CIIIi} Vehicle Stability (Side Friction)

Q_{SSDi} Stopping Sight Distance

Q_{Crri} Horizontal Alignment

Q_{VRRi} Vertical Alignment

Q_{wl} Driver Workload

The constituent risks for Curve No.57 are shown in Table 5.19.

Table 5.19 – Existing N76 Constituent Risk Ratings for Curve No. 57

Direction	Q_{CIi}	Q_{CIIi}	Q_{CIIIi}	Q_{SSDi}	Q_{Crri}	Q_{VRRi}	Q_{wl}
Forward (westbound)	0.090	0.111	1.000	1.000	1.000	1.000	1.000
Reverse (eastbound)	0.093	0.368	1.000	1.000	1.000	1.000	1.000

The values in Table 5.19 indicate that the risks associated with vehicle stability, stopping sight distance, horizontal alignment, vertical alignment and driver workload are all high.

Based on the above assessment and the findings of the site visit, it is considered that the following alignment issues are contributing to the higher than expected collision rates at Curve No.57.

- The tight radius of the curve in comparison with adjacent curves;
- The presence of a crest curve on the westbound approach to the curve that obscures the presence and severity of the horizontal curve;
- The low super-elevation rate on the curve.

It is considered that vehicles are travelling at a higher speed than would be expected for a horizontal curve with a radius if 163m, which is a considerably tighter bend than drivers would have negotiated on either approach. The presence of the crest curve in advance of the curve for westbound traffic is likely to cause drivers to fail to appreciate the presence or tightness of the bend, resulting in higher than expected vehicle speeds. Vehicles are then traversing the curve with a lower than expected super-elevation rate, providing a lower level of side friction, increasing the likelihood in the vehicle skidding and losing control, particularly in wet or damp conditions.

5.3.5 Potential Realignments on N76 Route

An indicative realignment scheme was developed for the highest collision risk, Curves No. 57. This indicative realignment scheme comprised an alignment with horizontal curvature relaxations of 3 steps below Desirable Minimum.

5.3.6 Analysis of Indicative Realignment

Curve 57 Realignment

An indicative realignment scheme was developed at this location. A modest horizontal realignment involved the replacement of Curve Nos. 56 and 57 and with a single 255m radius curve. A curve radius of 255m equates to a 3 Step below Desirable Minimum curve for a Design Speed of 100km/h in accordance with DN-GEO-03031. The realignment of the N76 using a reduced standard alignment at this location would require the realignment of 0.25 km of road and would likely not require the acquisition of any properties.

The indicative realignment also included a localised vertical realignment to the east of Curve 57 between Ch.6+057 and Ch.6+419. This alignment increases the length of the crest curve, increasing the k-value of the crest from 16 to 30. A k-value of 30 corresponds to 2-steps below Desirable Minimum for a design Speed of 100km/h, in accordance with GEO-DN-03031.

The combined effect of the horizontal and vertical realignments results in a total scheme length of 400m.

The extents of the indicative realignment at Curve No. 57 is shown on Figure 5.23.

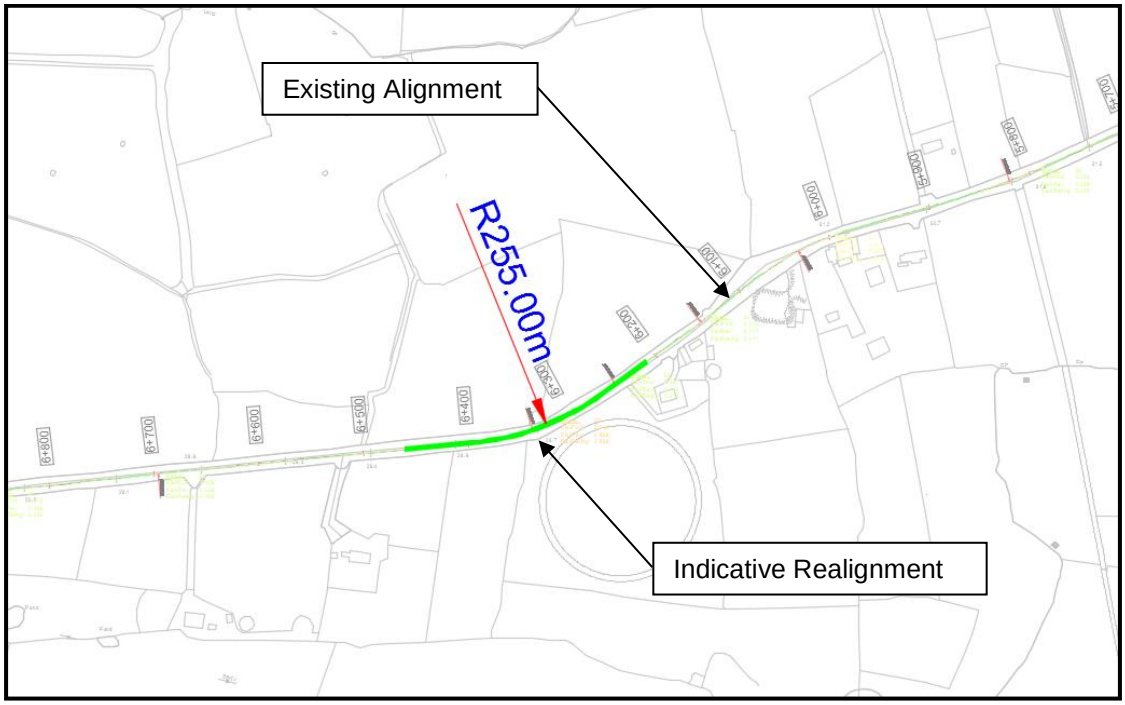


Figure 5.23 – N76 Curve 57 Realignment

The Risk Model was re-run using this realignment to demonstrate the impact on the Overall Risk Rating of site N76_M_2016_019 and the adjacent curves. The results of this analysis are shown in Table 5.20.

Table 5.20 – N76 Realignment Risk Ratings for Element No. 52 to No. 62

Alignment Element ID	Alignment Element Type	Curve Radius/m	Overall Risk Rating	Risk Order
52	Bend	261	0.546	7
53	Tangent		0.330	62
54	Bend	433	0.410	13
55	Tangent		0.330	63
56+57	Bend	255	0.372	31
58	Tangent		0.330	55
59	Bend	590	0.339	40
60	Tangent		0.331	53
61	Bend	355	0.409	16
62	Tangent		0.331	45

Curve No.57 Indicative Realignment Comparison

Table 5.21 – Comparison of N76 Alignment Risk Ratings for Element No. 52 to No. 62

Alignment Element ID	Alignment Element Type	Existing Alignment		Realignment	
		Overall Risk	Risk Order	Overall Risk	Risk Order
52	Bend	0.546	8	0.546	7
53	Tangent	0.330	60	0.330	62
54	Bend	0.411	15	0.410	13
55	Tangent	0.331	53	0.330	63
56	Bend	0.415	14	0.372	31
57	Bend	0.688	3		
58	Tangent	0.390	28	0.330	55
59	Bend	0.409	17	0.339	40
60	Tangent	0.331	46	0.331	53
61	Bend	0.438	10	0.409	16
62	Tangent	0.337	42	0.331	45

Table 5.21 shows a comparison of the Overall Risk Rating at Curve No.57. for the existing alignment and the realignment and indicates that the Overall Risk at Curve No.57 reduces from 0.688 to 0.372. Table 5.21 shows that the realignment provides a more consistent risk in the context of the adjacent elements.

5.3.7 N76 Alignment Assessment Findings

Overall Risk Reduction

On the basis of the assessment of the existing N76 alignment and the indicative horizontal and vertical alignments developed at the highest risk location, the Risk Model was re-run using the realignments at Curve No. 57.

Figure 5.24 shows a comparison of the existing Overall Risk Profile and the indicative realignment Overall Risk Profile. This shows a comparison of the before and after Risk Profiles once the highest risk section is improved.

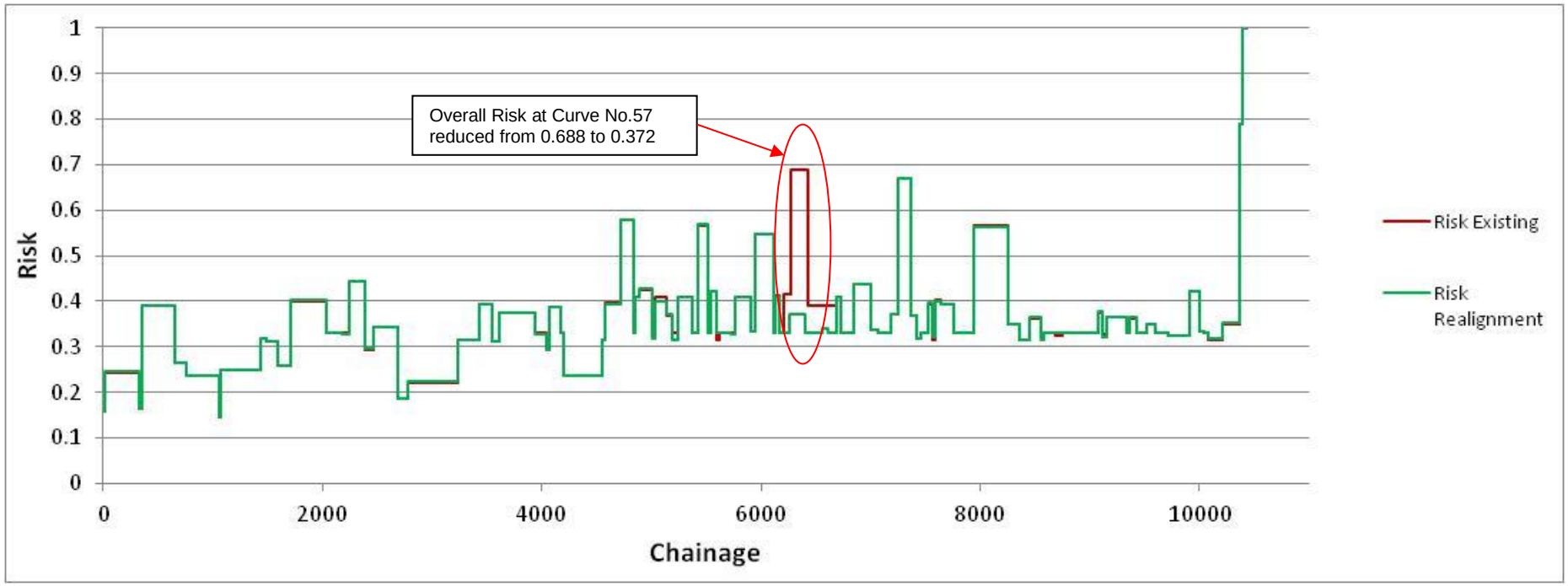


Figure 5.24 – N76 Comparison of Overall Risk Ratings

Speed Profile Comparison

The Speed Model shows an increase in Operating Speed from 79 km/h to 81 km/h at Curve No. 57.

The speed profile shows that the Average Operating Speed on the entire length of this section of the N76 remains constant at 89 km/h.

The Operating Speed along the existing N76 ranges from a high of 96 km/h to a low of 79 km/h, excluding the bends on the immediate approach to the Killheffernan Roundabout, equating to a speed variation along the route of 17 km/h, giving Consistency Rating of Fair. With the realignment at bend 57, the Operating Speed variation reduces to 15 km/h, equating to a Consistency Rating of Fair, consistent with the existing rating of Fair.

Figure 5.25 shows a comparison of the existing Operating Speed Profile and the indicative realignment Operating Speed Profile. This shows that the Operating Speed has increased slightly where indicative realignments have been modelled.

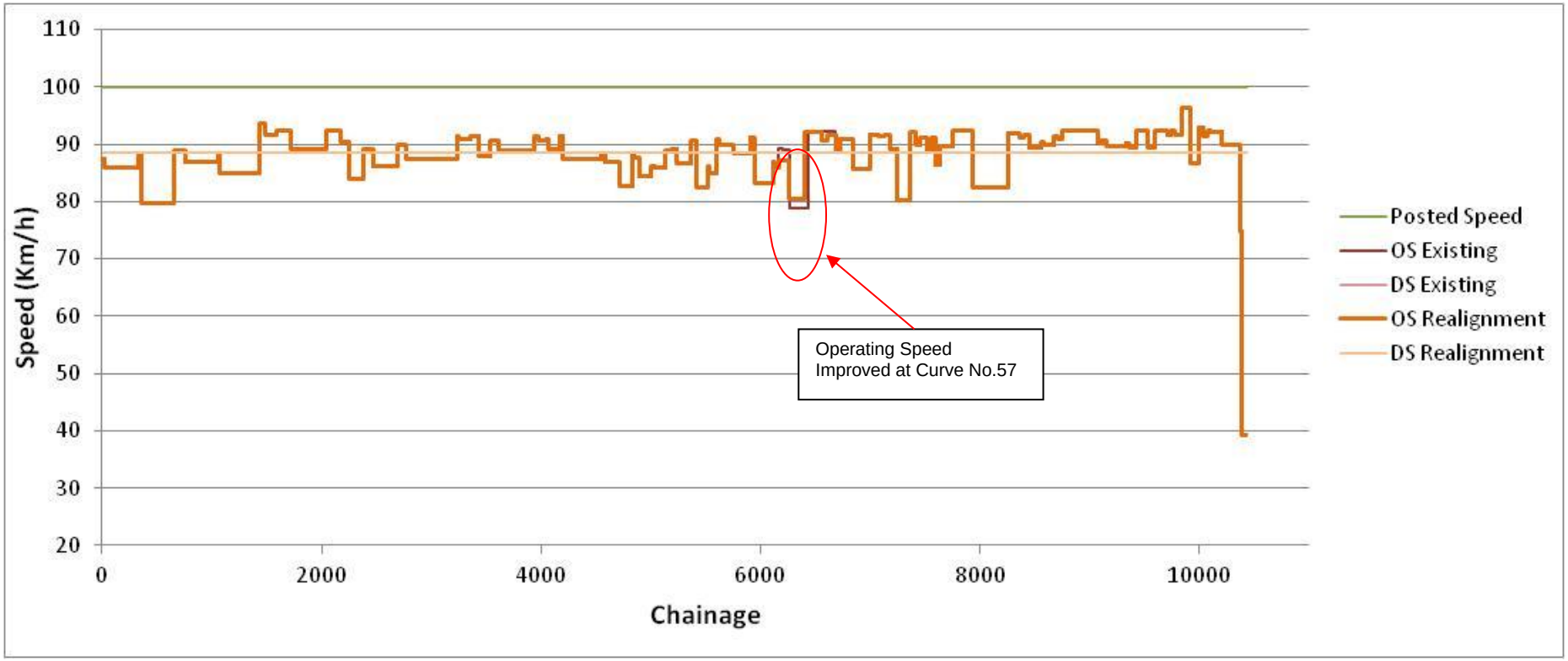


Figure 5.25 – Comparison of Operating Speeds

5.3.8 N76 Case Study Conclusions

1) The N76 at Seskin was selected for this case study due to concerns about recurring collisions at 3 locations, particularly site N76_M_2016_019.

2) 23.7% of the curves are below the standard for 100 km/h.

3) The Overall Risk Rating of the existing N76 ranges up to 0.688.

4) The Risk Model identifies site N76_M_2016_019 as being located on a bend with highest Overall Risk Rating.

5) The bend with highest Overall Risk Ratings coincides with the site of particular concern to Tipperary County Council and which has seen a number of recent collisions.

6) A review of the alignments at site N76_M_2016_019 and observations made on site indicates that there is a significant crest curve on the west bound approach to the site that obscures the presence of the horizontal bend from approaching drivers.

7) The super-elevation data from the routine Road Profiling surveys indicates that the super-elevation on the bend is 2%, well below the level of super-elevation that would be expected on a bend with a radius of 163m.

8) A combination of the tight radius, crest curve and low super-elevation is leading drivers to misjudge the severity of the bend resulting in excessive speeds, and increased risk of loss of control collisions.

9) An indicative realignment of 400m length would reduce the Overall Risk Rating from 0.688 to 0.372 at site N76_M_2016_019.

10) Operating Speed increases from 79 km/h to 81 km/h at the site.

11) Average Operating Speed for the N76 remains constant at 89km/h.

12) Speed Variation along the N76 falls from 17 km/h to 14 km/h.

13) Consistency Rating remains constant at Fair.

14) The improvement in Overall Risk Rating and Operating Speed is the consequence of two modest realignment schemes totalling 0.4 km (5.7%) of a 7 km long route.

15) An overall improvement strategy for the 7 km length of this sample route could tackle the next highest risk sites with ratings >0.6, or a larger set >0.5, to optimise the overall risk performance along this route.

6. CONCLUSIONS

The *RibGeom* project, based on bespoke and international research projects has obtained the following outputs:

1. A Risk Analysis Model capable of application at multiple scales (i.e. National, Regional, or Local).
2. Automated procedures & models to provide:
    a. Alignment definition (horizontal & vertical);
    b. Stopping Sight distance estimation;
    c. An Operating Speed Model;
    d. A Risk Model
3. Coupling of these models provides the means to:
    a. Perform Risk Screening exercises and develop roads needs studies at National and Regional levels;
    b. Optimise route planning (rolling programmes) and phasing of improvements to optimize:
        (ii) Collision Risk;
        (iii) Performance (Consistency);
        (iv) Cost.

RibGeom provides a proactive tool to assess Potential Realignments or other changes to the road geometry and to quantify both the inherent risk and optimise the resultant range of risk-reduction benefits that may be derived.

RibGeom model was developed utilizing detailed data from 30 reference sites on the National Road network. It was then applied to 3 case studies to evaluate the model outputs and to illustrate its potential use. The positive outcomes from these case studies demonstrate the ability of the *RibGeom* tool to select appropriate high-risk locations within extended lengths of typical legacy roads for which minor improvement schemes could be developed. Its benefits as a design aid to demonstrate appropriate application of the existing TII Road Design Standards can be implemented to achieve the greatest reduction of collision risk for modest scale works within a more consistent overall road alignment.

At the end of Phase 1 of this project the *RibGeom* tool is well developed for the analysis of the primary risk factors associated with the horizontal road alignment. Further refinement will be forthcoming for the vehicle stability factors associated with road surface friction, for which there is a separate parallel research and development programme underway by TII. The role of visibility also requires further refinement and specific investigation.

RibGeom can be used as a tool to inform Network Improvement Strategies through evaluation of Safety Benefits for various levels of investment. Baseline studies can be undertaken to evaluate the performance of the entire legacy national road network in terms of alignment quality, speed levels and consistency, and collision risks. Whereas these exercises would normally be very considerable undertakings in terms of the volume of data to be processed, through the use of, for example, alignment databases collected on the annual network surveys the requisite inputs are significantly reduced.

Ultimately the development of RibGeom may inform the enhancement of TII Design Standards for potential expansion of *DN-GEO-03030 Guidance on Minor Improvements to National Roads* to include new techniques and processes.

A scope of work for Phase 2 of the project has been developed to advance the work to date for the various headings and applications noted above.

With the *RibGeom* methodology Transport Infrastructure Ireland will adopt international best practice and latest research in a methodology that can achieve very significant improvements for road safety at affordable costs and in a targeted, consistent and environmentally sustainable manner. Thus, it would enable TII to make substantial progress towards the Strategic Goal for *"the provision, maintenance and operation of safe, efficient and sustainable networks of national roads"*.

7. FUTURE DEVELOPMENTS FOR *RIBGEOM* MODELS

7.1 Current Status of the *RibGeom* Models

The inter-related models which have been developed under *RibGeom* are, as they currently stand, capable of analysing – and providing insights into – existing roads and also proposed realignment schemes on these roads. Furthermore, the models demonstrate the potential of a risk-based approach to road design on the Irish road network. Nonetheless, it is important to note that at present they should be regarded as being at a "working prototype" stage (TRL 6), due to the following limitations:

- The calibration of the Risk Model was carried out using data from 30 pilot sites, the majority of which were located in the west of Ireland. This potentially introduces a bias, assuming that the character of these roads might vary from that of roads in other regions of the country.

- The Operating Speed Model has been calibrated using speed data from a relatively limited subset of nineteen of the pilot sites. More accurate calibration would require a greater quantity of data. As with the calibration of the Risk Model, the quality of the calibration could be improved by using data from a more geographically distributed set of locations.

- The Risk Model does not at present consider different traffic volumes between different routes. As such, it is currently only suitable for comparing different elements along the same route (with similar volumes); it is not currently suitable for comparison of elements from different roads with different traffic volumes. As it is likely that the collision risk does not vary as a linear function of traffic volume, further work would be required to determine the appropriate way of doing this.

- The Risk Model does not currently account for the influence of junctions along the length of the road.

7.2 Further Refinement of the *RibGeom* Models

The potential exists for further research to be carried out to establish the relationship between increasing traffic volumes and collision risk. This relationship is likely to be more complicated than simply assuming that the number of collisions will be directly proportional to the traffic volumes as vehicle interactions begin to affect average traffic speeds at higher volumes which can lead to distortion or dampening effects. A more accurate understanding of this relationship would enable the Risk Model to be used to compare different roads with different traffic volumes. This would in turn provide many benefits, for example:

- The risk-mitigating effects of proposed realignment schemes spread across different routes and in different regions of the country could be compared with each other. This would provide a valuable decision-making tool for better assessing the merits of proposed schemes and for prioritising the schemes which could provide the greatest

improvements in safety. This would in turn allow more efficient use of limited investment funding.

- The risk-effects of different alignment features (and different combinations of features) on collision risk could be analysed across the entire network. This could provide valuable data to inform future development of Design Standards and policies, as it would provide a means of assessing the real-world effects on collision risk of specific rules in the Standards, as well as that of specific Relaxations, etc.

7.3 Traffic Speeds and Collision Data

A consideration to note is that while the overall number of pilot sites used for calibration of the Risk Model was reasonably large (thirty), the relatively rare nature of collision events implied that the number of collision records from these thirty sites was nevertheless a somewhat small sample from a statistical point of view. Rather than attempting to improve on this by acquiring speed surveys from an even larger set of pilot sites, the potential exists for this to be achieved in a different way – to calibrate the risk model *from the entire network*. This would involve using the GPS data from the entire two-way single carriageway network in conjunction with the collision records for these same roads. This would have the potential to greatly increase the confidence level of the Risk Model results. However, as with the other items listed above, this would require the relationship between risk and traffic volumes to be determined first.

7.4 Dynamic Speed Profile Data

The Operating Speed Model has scope for future refinement, particularly with regard to modelling vehicle acceleration and deceleration processes. Recalibration of the model using a larger number of fixed-location speed surveys would be one option, but a preferable option might be to refine the model using continuous speed profile data, if this could be acquired cost-effectively. This kind of data, generally acquired by the use of GPS logging devices on board vehicles, would enable the speed profiles of a statistically significant number of vehicles to be analysed along entire routes, rather than at point locations. (Note that the GPS data used in this project is not suitable for this purpose, as the survey vehicles aim to travel at a constant speed of 60km/h). This would provide a more accurate source of data for modelling the operating speeds on tangents in particular.

A potential data source could be from the mobile phone operators. Currently the data from mobile phone GPS signals is collected by the telecommunications companies and processed to provide average speed and delay data through services such as *Google Traffic*. This data is made publicly available to indicate traffic delays, but there is great potential for other analyses. Examples are provided below of some current information from this source processed in different ways. It should be possible under suitable licence agreements to gain access to selected raw data to develop speed profiles along particular routes that will capture the variation arising from deceleration and acceleration to reflect the variable road alignment curvature.

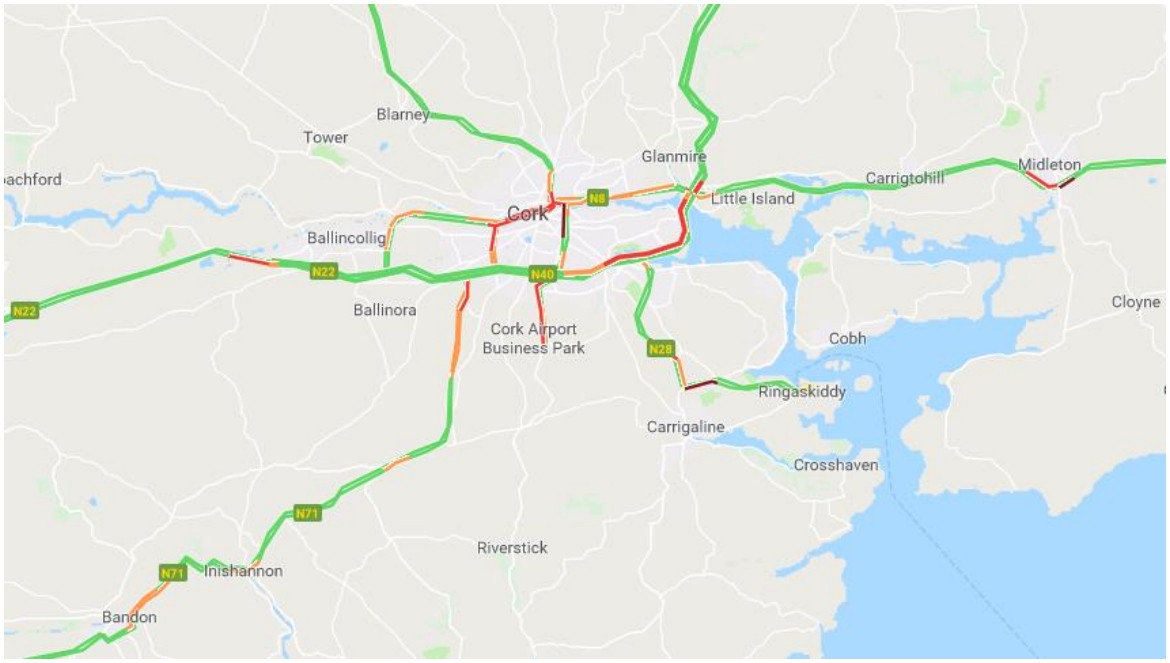


Figure 7.1 – Example of Google Traffic Data for Cork City Region (at PM Peak)

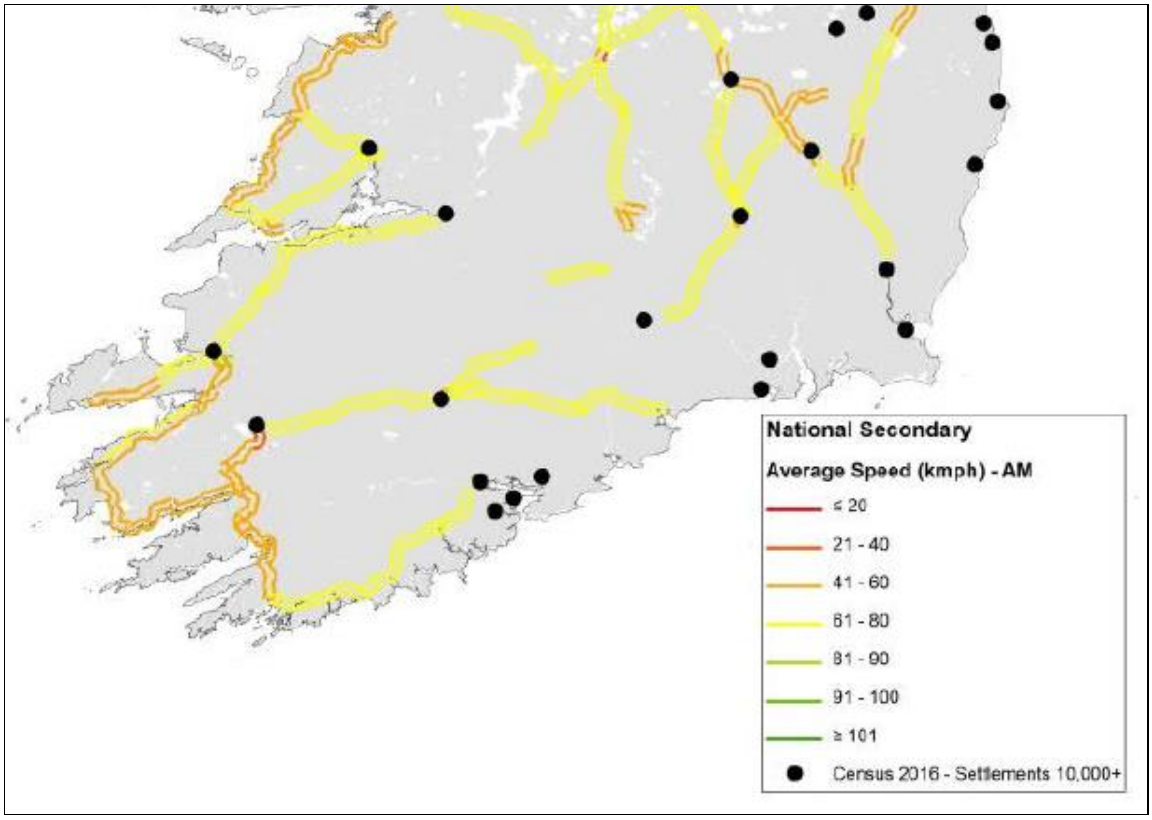


Figure 7.2 – Sample of Average Speeds on National Secondary Routes in the Southern Region (Source: Draft Report by Aecom for TII NR2040 Strategy)

7.5 Collision Risk at Junctions

The Literature Review which was conducted for this project (described in a separate report) determined that many of the methodologies which have been used for modelling risk on road links have tended to focus solely on the links between junctions without considering the effects of junctions themselves. This is because collisions in the vicinity of junctions are frequently caused by junction-specific factors which do not apply elsewhere on the links.

The Risk Model developed in this project employs a similar assumption. However, it is noted that the frequency of junctions along the Irish National Route network is probably greater than what would be typical in many other countries, even though the intensity of turning movements may be relatively low. As such, development of a methodology for incorporating the presence of junctions into the Risk Model would be beneficial.

7.6 Speed Limits Review

One further potential application of the model would be to provide a more rigorous method of determining the appropriate Design Speed values for the existing network (and for proposed realignments). This could also be extended to provide a more rigorous means of determining appropriate Posted Speed Limits for the National Route network, rather than the current regime of a default 100km/h speed limit in most non-urban locations.

APPENDIX A

LIST OF TECHNICAL NOTES

Risk Based Geometric Design

Appendix A

List of Technical Notes

Technical Note Number	Title
TN04	Literature Review
TN05	Pilot Site Inspections (in 5 groups)
TN5.1	N65, N66 & N52: Co. Galway & Tipperary
TN5.2	N67 & N85: Co. Clare
TN5.3	N59 & N63: Co. Galway
TN5.4	N5, N60, N61 & N83: Co. Mayo, Roscommon & Galway
TN5.5	N20 & N24: Co. Cork & Tipperary
TN06	Collision Data
TN10	Sensitivity Analysis
TN11	Model Description
TN12	Pilot Site Analysis

APPENDIX B

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