A new vehicle loading standard for road bridges in New Zealand
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Acknowledgements

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Executive summary

This report proposes a new vehicle loading standard for the design and evaluation of road bridges and other highway infrastructure in New Zealand.

With the progressive increase in heavy vehicle loadings associated with improved transport vehicle technology, the current vehicle loading standard for the design of road bridges in New Zealand, first introduced in 1972, is no longer appropriate for use in the design of future bridges and evaluation of existing bridges.

Because bridges have long lives, it is important that bridges built today are designed to match the heavy vehicle mass limits that may be economically viable in the long term. Research by Austroads for Australia's bridge design load review found that economically optimal mass limits were almost double existing mass limits and that it was worthwhile increasing bridge design loadings significantly to ensure that bridges built today will have the strength to carry all potential future vehicles. The benefits are very large and far greater than incremental bridge and pavement costs. An indicative economic assessment was carried out by applying the Austroads study findings to New Zealand. Subject to a number of assumptions, this shows that an increase in bridge design loads much higher than any mass limit increases that have been considered in previous studies is likely to be economically justified.

The provisions of the NZ Transport Agency ('the Transport Agency') *Bridge manual* have been reviewed and compared with other international design codes. After considering the outcome of this literature review, the authors formed the view that a new vehicle loading standard should be achieved by modifying either the existing *Bridge manual* or the Australian standard for bridge design AS5100. Therefore the development of the new vehicle loading standard has looked firstly at possible modifications to the *Bridge manual*, and secondly at how the AS5100 could be modified for the purpose.

The development of a new vehicle loading standard model has combined a 'top down' and a 'bottom up' approach. The top down approach considered the future freight need and likely configuration of vehicles to meet that need, and the bottom up approach was based on analysing the loading from current traffic. The top down approach was based on a truck and trailer combination freight vehicle with volume constrained loading. The bottom up approach considered weigh-in-motion data, a range of legally loaded vehicles, responses to an industry questionnaire on desired vehicle configurations, Transport Agency permit application vehicles and mobile cranes.

Loading models were developed based upon the *Bridge manual* HN-72 loading and the AS5100 SM1600 loading.

The report recommends a vehicle loading model that is 80% of the Australian SM1600 vehicle design loading be adopted for the design of new road bridges in New Zealand. The actions from the recommended loading model are a maximum of 50% greater than the current *Bridge manual* HN-HO loading.

The report also recommends an evaluation loading model of 40% of SM1600 for Class 1 loading, or 45% of SM1600 loading for HPMV loading.

The report also includes recommendations on axle and wheel loading, lane widths and number of lanes, multiple presence, dynamic load allowance, horizontal loads and load factors.

Abstract

This research report proposes a new vehicle loading standard for the design and evaluation of road bridges and other highway infrastructure in New Zealand. It is based upon a literature review of current traffic loading and bridge evaluation specifications in New Zealand and overseas, as well as a review of studies into the economic aspects of bridge design loadings and the economic benefits of increasing the mass limits of heavy vehicles in Australia and New Zealand.

The development of a new design loading standard took into consideration the future freight need and likely configuration of vehicles to meet that need, as well as analysing the loading from current traffic. Weigh-in-motion data, responses to an industry questionnaire on desired vehicle configurations, loading effects from a range of legally loaded vehicles, permit application vehicles and mobile cranes were all considered in determining the recommended design vehicle loading standard.

New evaluation loading standards are recommended for Class 1 and HPMV vehicle loading.

The report also includes recommendations on axle and wheel loading, lane widths and number of lanes, multiple presence, dynamic load allowance, horizontal loads and load factors.

1   Introduction

The primary purpose of this research was to determine a new vehicle loading standard for the design and evaluation of road bridges and other highway infrastructure in New Zealand.

The current vehicle loading standard for the design of road bridges in New Zealand was introduced in 1972. It is apparent that trucks are getting heavier and the stage has been reached where the current vehicle loading standard is no longer appropriate for the future. The introduction of high productivity motor vehicles (HPMVs) emphasises the need for a new vehicle loading standard to better cater for current and future vehicles.

New road bridges should be designed and built with sufficient strength to safely carry the vehicle loads they will be subjected to during their design life. Past experience internationally has shown that vehicle loads increase with time as transport technology improves, trucks become larger and more powerful, and economic activity (and hence the freight task) increases. Economic benefits flow from increased vehicle weights, as fewer trucks are required to transport a given weight of freight.

However, two main factors constrain increases in the design load for new road bridges. First, increasing the design load increases the construction cost of bridges. Second, the lower strength of existing bridges limits the extent to which legal weights can be increased, resulting in the greater strength of new bridges not being fully utilised until existing bridges are either progressively strengthened or replaced. Other constraining factors include pavement strength and truck safety. To determine the appropriate vehicle loading standard for the design of new bridges requires a balanced consideration of both the economic benefits of increased vehicle loading, and the engineering costs of constructing stronger bridges.

1.1   Vehicle loading standard for the design of new road bridges

In order to determine a new vehicle loading standard appropriate for the design of new road bridges in New Zealand the research included the following tasks:

- Preparation of a detailed literature review covering both the economic and engineering aspects of the project. This included a review of current vehicle load specifications and the development of bridge load models in New Zealand as well as in Australia, the USA, Canada, the UK and continental Europe.

- Qualitative assessment of the economic costs and benefits associated with the development of a new vehicle loading standard.

- Review and analysis of available weigh-in-motion (WiM) reports for New Zealand.

- Distribution of a questionnaire to industry (via the Axle Weights and Loadings Advisory Group) seeking input to the development of the new vehicle loading standard.

- Development of a new vehicle loading standard model using a 'top down' and 'bottom up' approach. The top down approach considered the future freight need and likely configuration of vehicles to meet that need and the bottom up approach was based on the effects of a range of current vehicle types operating in New Zealand, Australia and Europe.

Definition of a new vehicle loading standard entails defining more than the vehicle gravity loading, and the research has provided recommendations for axle and wheel loading, lane widths and number of lanes, multiple presence of vehicles, dynamic load allowance (DLA), horizontal loads and load factors.

1.2 Vehicle loading standard for the evaluation of existing road bridges

The evaluation loading is a design vehicle that is intended to replicate the structural actions from current legal vehicles, and can therefore be used in the assessment of bridges for current traffic loading. The research reviewed the applicability of the current evaluation loading, and proposed a new evaluation loading – expressed both in terms of the NZ Transport Agency (2013) *Bridge manual* HN-72 loading, and in terms of the recommended new vehicle loading standard.

1.3 Layout of this report

Chapter 2 of the report describes the current vehicle loading standard.

Chapter 3 provides the recommendations for a new vehicle loading standard, and chapter 4 compares the current and recommended loading models.

Chapter 5 presents the findings from the literature review.

Chapter 6 presents the findings from the economic analysis.

Chapter 7 explains the development of the recommended vehicle loading standard, and incorporates the findings from the WiM analysis and the stakeholder communication.

These seven chapters comprise the body of the report. They are supported by detailed appendices as follows (note that some appendices include annexes):

- Appendix A: Literature review

- Appendix B: Economic analysis

- Appendix C: Weigh-in-motion analysis

- Appendix D: Model development

- Appendix E: Stakeholder consultation

- Appendix F: Glossary.

2   Current vehicle loading standard

2.1   NZ Transport Agency (2013) *Bridge manual*

The current version of the *Bridge manual* (NZ Transport Agency 2013) specifies that the traffic loading for gravity effects is HN-HO-72. This includes an element to represent the normal loading (HN) and an element to represent an overloading (HO).

HN loading consists of:

- 3.5kPa uniformly distributed lane load which equates to a 10.5kN/m lane load for the specified lane width of 3m

- 2 x 120kN axle loads at a fixed axle spacing of 5m and a transverse wheel spacing of 1.8m

- 60kN wheel load over a wheel contact area of 500mm x 200mm

- a DLA for moment in simple or continuous spans of 0.3 for spans from 0m to 12m, reducing to 0.11 at 100m span

- a DLA for all other actions of 0.3

- a serviceability limit state (SLS) load factor of 1.35.

HO loading consists of:

- 3.5kPa uniformly distributed lane load which equates to a 10.5kN/m lane load for the specified lane width of 3m

- 2 x 240kN axle loads at a fixed axle spacing of 5m and a transverse wheel spacing of 1.8m

- 120kN over a wheel contact area of 900mm x 600mm, or 240kN over a wheel contact area of 3000mm x 200mm

- a DLA for moment in simple or continuous spans of 0.3 for spans from 0m to 12m, reducing to 0.11 at 100m span

- a DLA for all other actions of 0.3

- a SLS load factor of 1.0.

HN and HO loading are represented diagrammatically in figure 2.1.

Figure 2.1   *Bridge manual* **HN-HO loading**
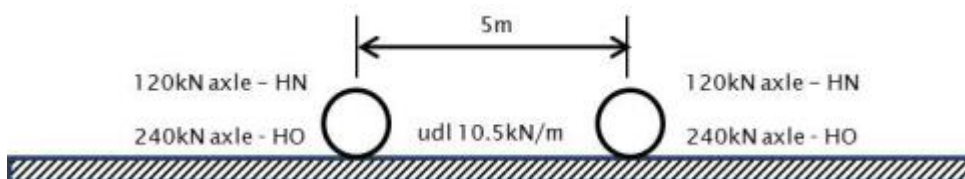
2.2 Australian bridge design standard AS5100

The current version of the Australian bridge design standard AS5100 (SA 2004) specifies SM1600 as the traffic loading for gravity effects. This includes the W80 wheel load, the A160 axle load, the M1600 moving traffic load, and the S1600 stationary traffic load.

SM1600 loading consists of:

- W80 wheel load which is an 80kN wheel load over a wheel contact area of 400mm x 250mm

- A160 axle load which is a 160kN axle

- M1600 moving load which is a 6kN/m uniformly distributed lane load together with four tri-axle sets of 360kN each

- S1600 stationary load which is a 24kN/m uniformly distributed lane load together with four tri-axle sets of 240kN each

- a DLA of 0.4 for W80 and A160, 0.35 for the tri-axle set in M1600, 0.3 for the complete M1600, and 0.0 for S1600

- a SLS load factor of 1.0.

M1600 and S1600 loading are represented diagrammatically in figure 2.2.

Figure 2.2   AS5100 M1600 and S1600 loading
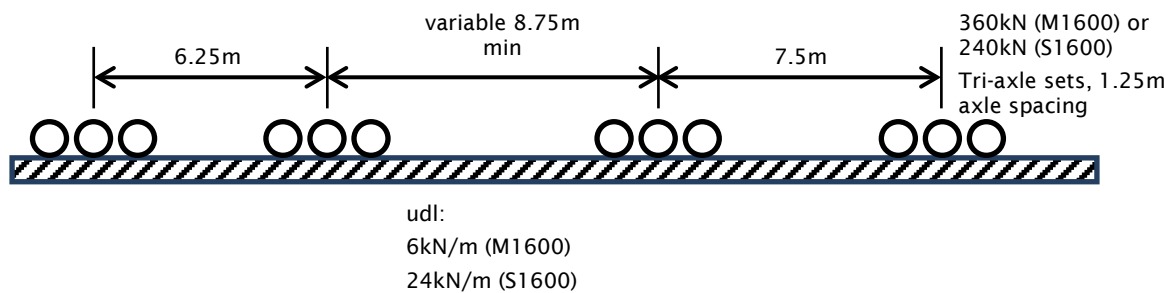
3   Recommended vehicle loading standards

The primary purpose of this research was to determine a new vehicle loading standard for the design and evaluation of road bridges and other highway infrastructure in New Zealand. The researchers formed the view that a new loading standard for New Zealand should be achieved by modifying either the existing *Bridge manual* or the AS5100. Implementing a new loading model that was not based on an existing standard would be inefficient, and if the existing *Bridge manual* loading could not be modified for the purpose, then the Australian standard should be adapted, in line with the high-level objective of harmonising standards between New Zealand and Australia.

Therefore the development of the loading model for a new vehicle loading standard looked first at possible modifications to the *Bridge manual*, and second at how the AS5100 could be modified for the purpose.

3.1   Recommended design vehicle loading model

To allow for future freight needs and a range of heavy vehicles, the design vehicle loading model should be based on the AS5100 and consist of:

- W80 wheel load

- A160 axle load

- 80% of the M1600 moving load

- 80% of the S1600 stationary load

- a DLA of 0.4 for W80 and A160, 0.35 for the tri-axle set in M1600, 0.3 for the complete M1600, and 0.0 for S1600

- a SLS load factor of 1.0.

This loading model is referred to in this report as 0.8_SM1600. The moving and stationary components are represented diagrammatically in figure 3.1. The SLS actions from the recommended loading model are a maximum of 50% greater than the current *Bridge manual* HN-HO loading (with the SLS factor of 1.35).

Figure 3.1   Recommended design vehicle loading model – 0.8_SM1600
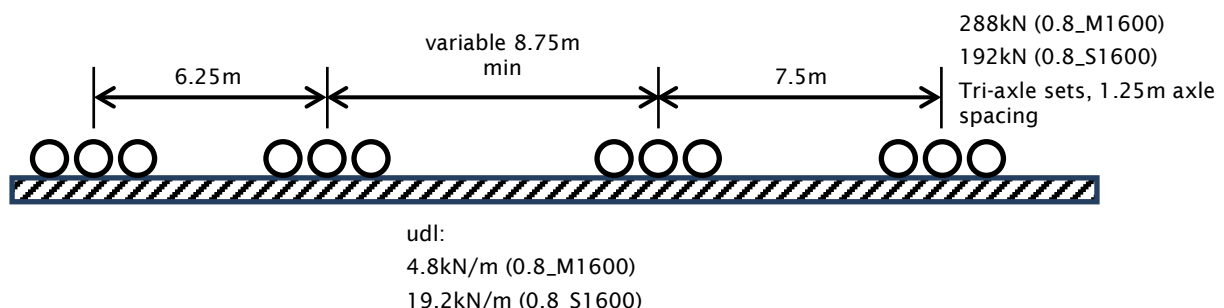
An alternative design load model based on modification of the *Bridge manual* was developed, but is not the recommended model. The design vehicle loading model to allow for future freight needs and a range of heavy vehicles would be 2.0_HN-72 (with a SLS load factor of 1.0). The SLS actions from this loading model are also a maximum of 50% greater than the current *Bridge manual* HN-HO loading (with a SLS factor of 1.35 included).

The AS5100 is preferred as the basis for the recommended design vehicle loading model, rather than the *Bridge manual*, because the SM1600 loading model, with its multi-axle configuration, provides a more realistic representation of the magnitude and distribution of the axle loads on bridges that result from multi-axle combination vehicles that are becoming the predominant freight vehicle. To represent these vehicles using the two-axle HN loading model it is necessary to increase the axle loads to unrealistically large values. This causes anomalies in the design for short spans (where axle loads dominate) and when designing for local effects (for example in deck slab design). The researchers believe that design is better served by the logically consistent set of loads in the SM1600 model which can be applied to the design of all bridge elements.

The report also includes recommendations on lane widths and number of lanes, multiple presence, DLA, horizontal loads and load factors. These are contained in appendix D5.

3.2   Recommended evaluation vehicle loading model

The evaluation vehicle loading is a design vehicle that is intended to replicate the structural actions from current legal vehicles, and can therefore be used in the assessment of bridges for current traffic loading. Notwithstanding that the current evaluation loading has served New Zealand well, if the evaluation loading is to encompass a complete range of spans and design actions the researchers recommend that a new evaluation loading be adopted. Consistent with the recommendation for the design vehicle loading model, the recommended evaluation vehicle loading model should be based on the AS5100.

The recommended evaluation vehicle loading model for Class 1 legal vehicles is 0.40_SM1600 and consists of:

- 40% of the M1600 moving load

- 40% of the S1600 stationary load

- 100kN axle load

- a DLA of 0.4 for the axle load, 0.35 for the tri-axle set in M1600, 0.3 for the complete M1600, and 0.0 for S1600.

The recommended evaluation loading model for HPMV legal vehicles is 0.45_SM1600 and consists of:

- 45% of the M1600 moving load

- 45% of the S1600 stationary load

- 100kN axle load

- a DLA of 0.4 for the axle load, 0.35 for the tri-axle set in M1600, 0.3 for the complete M1600, and 0.0 for S1600.

Alternative evaluation load models based on modification of the *Bridge manual* were developed, but are not the recommended models. These are 1.0_HN-72 for Class 1 legal loading and 1.1_HN-72 for HPMV legal loading.

3.3   Summary of recommended vehicle loading models

The recommended vehicle loading models are based on the AS5100. Alternative vehicle loading models based on modification of the *Bridge manual* were developed, but are not the recommended models.

Table 3.1   Recommended vehicle loading models

	Recommended loading model	Alternative loading model
Design vehicle loading model to allow for future freight needs and a range of heavy vehicles	0.8_SM1600 (includes W80 wheel load and A160 axle load)	2.0_HN-72
Evaluation vehicle loading model for Class 1 legal vehicles	0.40_SM1600 (includes 100kN axle load)	1.0_HN-72
Evaluation vehicle loading model for HPMV legal vehicles	0.45_SM1600 (includes 100kN axle load)	1.1_HN-72

4 Comparison between current and recommended vehicle loading standards

4.1 Design vehicle loading model

The current design vehicle loading model is the *Bridge manual* HN-HO loading. The recommended design vehicle loading model is 0.8_SM1600. The latter allows for future freight needs and a range of heavy vehicles, and represents a significant increase on the current model. This is illustrated in figure 4.1, where the sagging moment for a 100m simply supported span is 43% greater in the recommended loading model.

However, because the recommended loading model is based on the multi-axle configuration of the Australian bridge design standard, the difference between the current and recommended loading models is not so pronounced for short span simply supported bridges. Figure 4.2 plots the same sagging moment information up to 30m span. For spans below 20m the moments due to the recommended model are slightly less than the current HN-HO loading, increasing to 21% more at 30m span. Therefore the impact of the recommended design loading model will not be significant for short-span simply supported bridges, which comprise the majority of the existing bridge stock.

Figure 4.1   Comparison of current and recommended design vehicle loading models – to 100m span
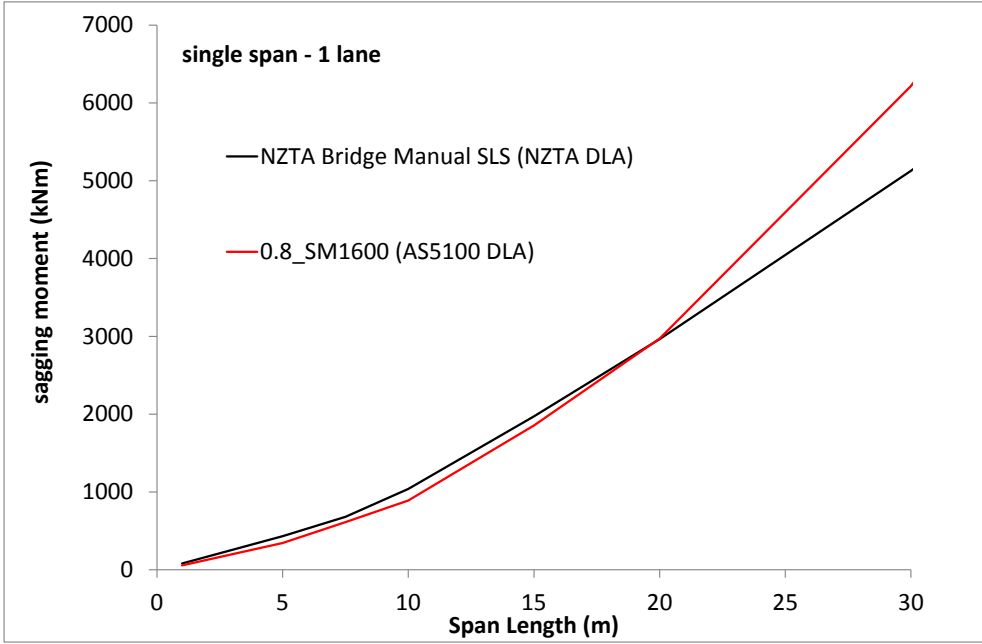
4.2   Evaluation vehicle loading model

The current evaluation vehicle loading model for Class 1 legal loading is 0.85_HN-72. Appendix D7 of this report is a review of this evaluation loading. The 0.85_HN-72 loading was originally proposed to represent Class 1 legal loading for simply supported spans up to 35m; however, the design actions from the 0.85_HN-72 loading are less than some structural actions from Class 1 legal vehicles over a wide range of spans. An evaluation loading of 0.40_SM1600 is recommended for Class 1 vehicles.

The recommended evaluation load is an increase over the current evaluation load for most spans and design actions. This will lead to existing bridges having a reduced load rating if they are re-rated against 0.40_SM1600. However, for the sagging moment in simply supported bridges in the span range from 10 to 30m, the difference between 0.40_SM1600 and 0.85_HN-72 is modest. This is plotted in figure 4.3, and shows that the design actions from 0.40_SM1600 are between 11% greater and 15% less than the design actions from 0.85_HN-72. Therefore the impact of the recommended evaluation loading model will not be significant for short-span simply supported bridges, which comprise the majority of the bridge stock.

Figure 4.3   Ratio of recommended to current evaluation models – sagging moment up to 30m span

single span - 1 lane
NZTA DLA applied to HN
AS5100 DLA applied to SM

——(0.40SM/0.85HN) sag
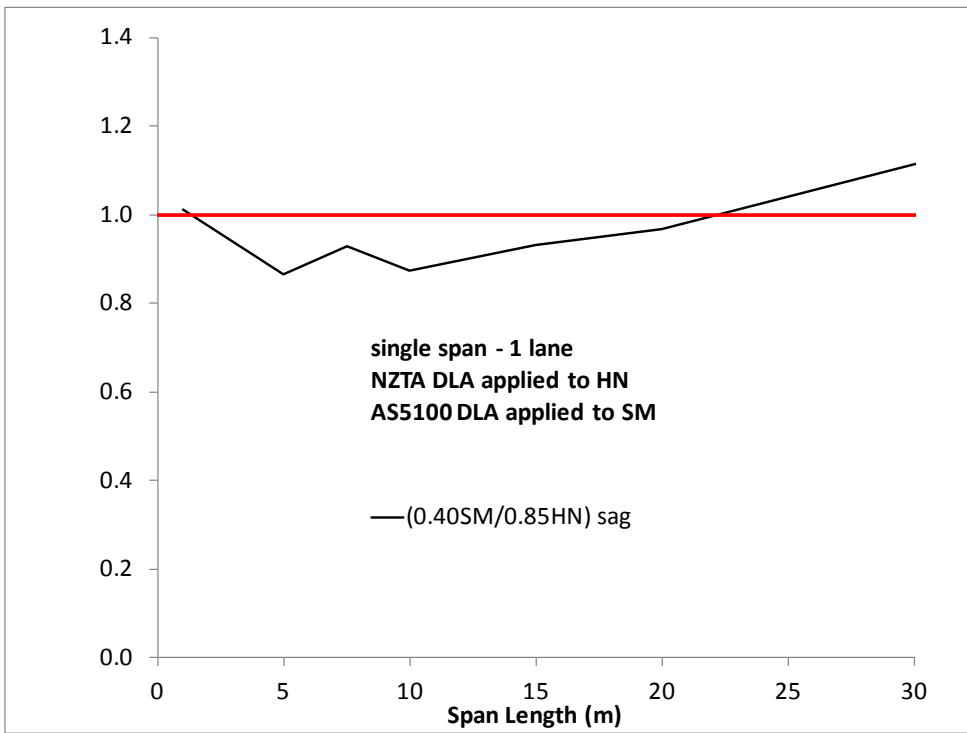
5   Findings from the literature review

A literature review (see appendix A) was undertaken covering both the economic and the engineering aspects of changing the vehicle loading standard. This included a review of current vehicle load specifications and a review of the development of bridge load models in New Zealand as well as in Australia, the USA, Canada, the UK and Europe.

The review indicated that the current New Zealand traffic loading standard for highway bridges is towards the middle of the range in terms of magnitude of the load when compared with other international design codes for all span lengths. This is illustrated in figures 5.1 and 5.2. Figure 5.1 is the serviceability limit state (SLS) sagging bending moment (with DLA included) for one traffic lane, and figure 5.2 is the same information at the ultimate limit state (ULS) for three lanes loaded. New Zealand loading is low in comparison with other international codes at SLS and towards the middle of the range at ULS. In these figures the sagging moment is divided by span$^{1.5}$ as this provides a better visual representation of the moment across the complete span range.

The recommended design vehicle loading standard of 0.8_SM1600 is also plotted on these graphs.

Figure 5.1   Comparison of SLS sagging moment for a single lane loaded (DLA included)
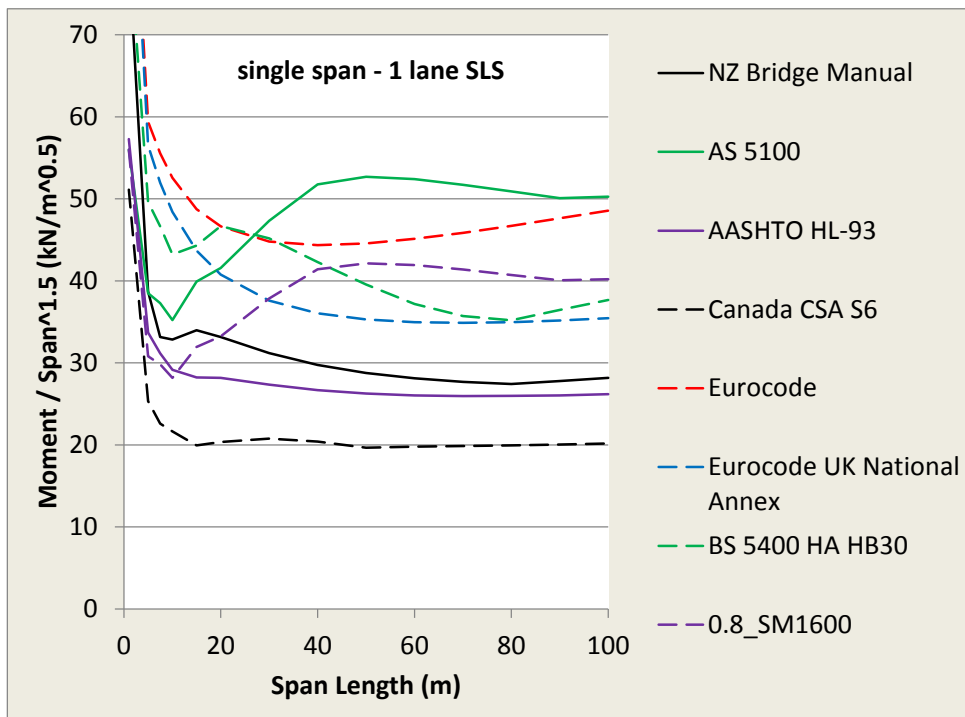
Figure 5.2   Comparison of ULS sagging moment for three lanes loaded (DLA included)
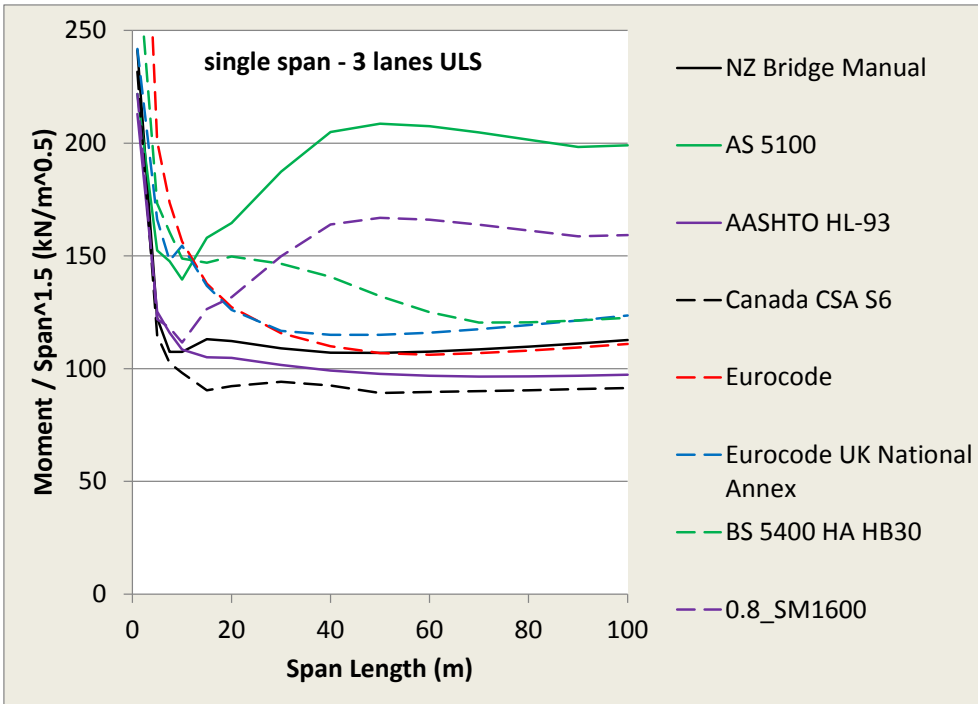
Based on the findings of the literature review and comparisons made between the *Bridge manual* and other international design codes, it was concluded that the following be considered in the development of the new vehicle loading standard for New Zealand:

- The traffic loading model should include a uniformly distributed load in conjunction with a design vehicle.

- The SLS load factor for the traffic loading model should be adopted as 1.0.

- To address the design of short spans, the traffic loading model should include a single axle loading to represent an individual overloaded axle.

- The wheel loading pressure should be consistent with the other design codes.

- The step increment for additional design lanes should be less than or equal to the standard traffic lane width.

- The DLA should be reassessed to be consistent with other international codes.

- The specification of the braking and centrifugal forces should be retained as a proportion of the weight of the traffic loading.

- The live load factor at the ULS appears to be adequate for current bridge loadings. However, this is subject to economic evaluation and any allowances made for future increases to bridge loadings. It is recommended that the live load factor at the ULS should be chosen to achieve similar or greater design action effects as the current standard. However, it is recommended that the factor is chosen to achieve design actions which are not less than the current standard.

- The current procedure for evaluation should be retained. The magnitude of the live load to be used in the evaluation should be reviewed once the new vehicle loading model has been developed.

These recommendations were subsequently addressed in the development of the recommended loading model.

6   Economic analysis

A high-level economic analysis of the impact of increasing the vehicle loading standard was undertaken, and is presented in appendix B.

Because bridges have long lives, it is particularly important that bridges built today are designed to match the heavy vehicle mass limits that might potentially be economically viable in the long term.

Research by Austroads in 1996 for Australia's bridge design load review (Austroads Project RUM.H.96, refer section A3.2.2 of this report) found that economically optimal gross and axle mass limits were almost double existing mass limits. This study concluded that it was worthwhile increasing bridge design loadings significantly to ensure that bridges built today will have the strength to carry all potential future vehicles. The benefits are very large and are far greater than any incremental bridge and pavement costs.

Austroads found that bridge construction costs are not highly sensitive to increases in design vehicle mass, increasing approximately 1% for each 10t increase in axle set loading.

An indicative economic assessment was carried out by applying the Austroads study findings to New Zealand and, subject to a number of assumptions, this shows that a substantial increase in bridge design load is likely to be economically justified. An increase in bridge design load that accommodates an increase in maximum axle set weight of up to 10t is only expected to increase bridge construction costs by approximately 1% and would result in a benefit–cost ratio of 6.6. Such an increase in design load is much higher than any mass limit increase that has been considered in previous studies.

The economic analysis concluded that because significantly higher bridge design loads only require a slight increase in bridge construction costs it would be worth considering a substantial increase in New Zealand's bridge design load to future-proof new and replacement bridges for all potential future heavy vehicle configurations and loads over the next 30 years, not just the types of HPMV implemented under the Land Transport Rule: Vehicle Dimensions and Mass Amendment 2010 (VDAM Rule Amendment 2010).

7 Development of the recommended vehicle loading standard

The loading model was developed for single and continuous bridges with spans up to 100m. The development of the recommended vehicle loading standard is described in appendix D.

Bridge loading standards around the world vary in the way that they define the design vehicle loading, and how they prescribe the types of load to be designed for. After considering the outcome of the literature review, which reviewed six loading standards from around the world, the researchers formed the view that a new loading standard for New Zealand should be achieved by modifying either the existing *Bridge manual*, or the AS5100. The basis for this view was that:

- Implementing a new loading standard that was not based on an existing standard would be inefficient, particularly with regard to the development of software that is very much a part of modern bridge design and assessment.

- If the existing *Bridge manual* could not be modified to meet the requirements of a new vehicle loading standard, modifying an international standard other than from Australia would be contrary to the high level objective of harmonising standards between New Zealand and Australia.

Therefore, the development of the loading model for a new vehicle loading standard looked firstly at possible modifications to the *Bridge manual*, and secondly at how the AS5100 could be modified for the purpose.

The development of a new vehicle loading standard model combined a 'top down' and a 'bottom up' approach. The top down approach considered the future freight need and likely configuration of vehicles to meet that need, and the bottom up approach was based upon analysing the loading from current traffic. This approach is shown schematically in appendix D, figures D.1 and D.2.

The top down approach was based on a truck and full trailer combination freight vehicle with volume constrained loading at a density of 0.73t per cubic metre. Refer to appendix D for an explanation of this approach. The bottom up approach considered weigh-in-motion data, a range of legally loaded vehicles, responses to an industry questionnaire on desired vehicle configurations, NZ Transport Agency permit application vehicles and mobile cranes.

Loading models were developed based upon the *Bridge manual* HN-72 loading, and the AS5100 SM1600 loading. To ensure that the loading model will allow for future freight needs and a range of heavy vehicles, the recommended loading model is either 2.0_HN-72 or 0.8_SM1600. For reasons outlined in appendix D the researchers recommend that the loading model of 0.8_SM1600 is adopted.

Appendix D also includes recommendations on axle and wheel loading, lane widths and number of lanes, multiple presence, DLA, horizontal loads and load factors.

The evaluation loading is a design vehicle that is intended to replicate the structural actions from current legal vehicles, and can therefore be used in the assessment of bridges for current traffic loading. Notwithstanding that the current evaluation loading has served New Zealand well, if the evaluation loading is to encompass a complete range of spans and design actions the researchers recommend that a new evaluation loading be adopted. Consistent with the recommendation for a loading model of 0.8_SM1600, it is recommended that an evaluation loading of 0.45_SM1600 is adopted to cover legal vehicles including HPMVs. A reduced evaluation loading of 0.40_SM1600 is appropriate for Class 1 loading.

8   References

New Zealand Transport Agency (2013) *Bridge manual*, 3rd edition with amendments. Wellington: NZ Transport Agency.

Standards Australia (SA) (2004) *AS5100 Bridge design.* Sydney.

Appendix A: Literature review

Contents

A1   Executive summary

With the progressive increase in heavy vehicle loadings associated with improved transport vehicle technology, the current vehicle loading standard for the design of road bridges in New Zealand, first introduced in 1972, may no longer be appropriate for use in the design of future bridges and evaluation of existing bridges.

A review of the current traffic loading and bridge evaluation specifications in New Zealand and overseas has been undertaken. The background to the development of these traffic loading models and other specifications is presented.

A review of studies into the economic aspects of bridge design loadings and the economic benefits of increasing the mass limits of heavy vehicles in Australia and New Zealand has also been undertaken.

An economic study carried out in Australia at the time of developing the *Australian standard for bridge design* AS5100 found that existing bridge assets and design loading of the standard at the time (T44 loading) were causing limitations to transportation productivity and restricting economic benefits. It was found that it would be economic to increase the traffic loading of bridges significantly to provide for the heaviest anticipated future vehicles and invest in increasing the capacity of existing and new bridges.

A study into the economic benefits of increasing the load and dimension limits for heavy vehicles in New Zealand found that benefits would arise from increased productivity. However, the study also concluded that the cost of corner widening of roads, which were significantly larger than the cost to improve bridges and pavements, overwhelmed any economic benefits. As a result, further studies were carried out where it was found that economic benefits would arise from operating the existing vehicle fleet at heavier weights or by increasing the vehicle dimensions and weights on selected key routes. This led to the introduction of high productivity motor vehicles (HPMVs) to the road network in 2010. Economic studies on the introduction of HPMVs or increasing the weight limit from 44t to 50t found benefits to increased productivity in the order of 16%. However, it was also found that nearly half of the HPMV permit applications were declined, mainly due to bridge limitations, which suggests that bridge assets are currently restricting potential economic benefits.

The provisions of the *Bridge manual* have been reviewed and compared with other international design codes. This review concludes that the design traffic loading in New Zealand is towards the middle of the range in terms of magnitude of the load and tends to follow the trends of the loading from the American Association of State Highway and Transportation Officials (AASHTO). Previous reviews of the *Bridge manual* concluded that the traffic loading was insufficient at the serviceability limit state (SLS); however it was adequate at the ultimate limit state (ULS). This has resulted in a proposed increase, in the third edition of the manual (NZ Transport Agency 2013), to the SLS live load factor from 1.35 to 1.5, specifically to accommodate HPMVs. This proposed SLS load factor is a departure from trends of other design codes where 1.0 is typically used. It is the intention of this project to determine a new traffic loading model which results in an SLS live load factor of 1.0, thus requiring an increase to the existing unfactored traffic loading model.

The review of other international design codes found significant differences in the magnitude of the traffic loading models, with the Australian and European codes being the highest and the North American codes the lowest. The common trend of the traffic loading models is to nominate both a uniformly distributed load and a design vehicle. The traffic loading models for these international codes have been derived from the weigh-in-motion (WiM) data collected.

The procedure for evaluation of bridges in the *Bridge manual* is consistent with other design codes. However, the magnitude of the traffic loading used in the evaluation should be reconsidered.

A2   Introduction

New road bridges should be designed and built with sufficient strength to safely carry the vehicle loads that they will be subjected to during their design life. Past experience has shown that as transport technology and economic activity (and hence the freight task) increase, vehicle loads also increase and trucks become larger and more powerful. Economic benefits flow from increased vehicle weights, as freight volume per vehicle increases and fewer trucks are required to transport a given weight of freight. However, two main factors constrain increases in the design load for new road bridges. First, an increase in the design load results in an increase in the construction cost of bridges. Second, the lower strength capacity of the existing bridge infrastructure constrains the extent to which truck weights can be increased, resulting in the greater strength capacity of the new bridges not being utilised to their full potential without extensive (and expensive) strengthening of the existing infrastructure. Other factors include pavement strength and truck safety. To determine the appropriate vehicle loading standard for the design of new bridges requires a balanced consideration of both the economic benefits of increased vehicle loading, and the engineering cost of increased vehicle loading on both new and existing bridges.

The magnitude of the vehicle loading used for the assessment of existing bridges to carry current traffic should not be the same as that used for the design of new bridges. Such an approach will lead to conservative bridge assessments and unnecessary strengthening or load restrictions. There are two main reasons why this is so:

• Vehicle loading for the design of new bridges to achieve a 100-year design life must allow for future increases in vehicle weights (as discussed above) and so should be greater than the loading from current vehicles.

• Vehicle loading from current vehicles is known with greater certainty than the range of vehicle weights that a bridge may be subjected to over its 100-year design life, and so the load factor applied to current vehicles can be less than that applied to the vehicle loading for the design of new bridges.

The current vehicle loading standard for the design of road bridges in New Zealand was first introduced in 1972. It is apparent from observation and WiM data that vehicle loads are increasing. A previous amendment to the *Bridge manual* in 2004 included the increase of the serviceability live load factor from 1.0 to 1.35 to accommodate the increase of vehicle loading. The introduction of HPMVs to the network in 2010 also represents a further increase to the vehicle loadings with a proposed increase to the serviceability live load factor to 1.5 as specified in the third edition of the *Bridge manual*.

This literature review considers previous studies carried out in New Zealand in relation to vehicle loading on bridges in New Zealand. A review of certain aspects of the *Bridge manual* related to traffic loading has been carried out and compared with trends of other international bridge design codes and their development of traffic loading models to represent the traffic stream within those regions.

There are two ways of carrying out evaluation assessments to determine if a bridge has the capacity to safely carry a nominated vehicle. One approach is to compare calculated structural actions due to loading from the nominated vehicle with the calculated capacities of the bridge (capacity approach). The other approach is to compare the calculated structural actions due to the loading from the nominated vehicle with the calculated structural actions due to the loading from another vehicle which the bridge has been previously assessed as safe to carry (reference vehicle approach).

The literature review considers bridge load models for design and evaluation from the following countries:

New Zealand	*Bridge manual*, 3rd ed (NZ Transport Agency 2013).
	Overweight permit manual (NZ Transport Agency 2005).
Australia	*AS5100 Australian standard for bridge design* (SA 2004) with subsequent amendments.
USA	*AASHTO LRFD bridge design specification*, 5th ed (AASHTO 2010).
	The manual for bridge evaluation, 2nd ed (AASHTO 2011).
Canada	*CAN/CSA-S6-06 Canadian highway bridge design code* (CSA 2006).
Europe	*EN 1991-2: Actions on structures – part 2: traffic loads on bridges* (EN 1991-2) (CEN 2003).
	EN 1990:2002+A1:2005: Basis of structural design (EN 1990: 2002+A1) (CEN 2005).
United Kingdom	*National annex to British standard BS EN 1991-2: 2003* (BSI 2008).
	National annex to British standard BS EN 1990:2002+A1:2005 (BSI 2009).
	Interim advice note 124/11: Use of Eurocodes for the design of highway structures (Highways Agency 2011a).
	BD 21/01: The assessment of highway bridges and structures (Highways Agency 2001a).
	BD 86/11: The assessment of highway bridges and structures for the effects of special types general order (STGO) and special order (SO) vehicles (Highways Agency 2011b).

These documents are considered to be the most relevant and comprehensive and applicable to the New Zealand situation. Details of the basis for the specific bridge load models are described.

Separate recommendations will be made for the design loading and the evaluation loading, and this literature review addresses each of these loadings separately.

The load models adopted for fatigue in the above documents are not described as fatigue is included in the associated research project ART 2 Fatigue design criteria for road bridges in New Zealand.

The structural performance of a bridge can be measured in terms of reliability, or probability of failure. A reliability analysis requires the compilation of probabilistic load and resistance models. These resistance models are dependent on the different structure types, material properties and considerations of parameters such as flexural and shear capacity. To determine the load factors for bridges used in the load combinations in some design codes a calibration procedure is carried out to achieve a reliability index, β-factor, which is close to a predetermined target reliability index, which is typically 3.5. This calibration procedure is an extensive exercise and requires the selection of representative bridges, establishment of a statistical database for load and resistance parameters, development of probabilistic strength models, and includes material tests and field measurements. However, the procedure was not within the scope of this research.

A high-level qualitative review of the economic aspects of increasing bridge design loadings was also carried out. The review of economic considerations in New Zealand looked at studies into increasing weight limits for the road network, the background to the introduction of HPMVs and the associated cost

impacts to infrastructure to increase the weight limits. Relevant aspects of the NZ Transport Agency (2010) *Economic evaluation manual* (EEM) were also noted. A review of the economic factors considered in the development of the Australian bridge design loads was included where these factors had a heavy influence in the derivation of the traffic loading. The review of economic aspects in Australia provided a useful context for discussion of the economic aspects in New Zealand.

A3 Economic considerations

A3.1 Introduction

This section reviews relevant previous studies and reports on economic aspects of bridge design loadings. The main focus is on Australia and New Zealand as both countries have conducted relatively recent investigations into the economic benefits of higher mass limits and in the case of Australia, higher bridge design loads specifically. The Australian studies are discussed first as they provide useful context for some of the discussion in the New Zealand section.

Approximately 20 reports and studies were examined for this part of the literature review. Rather than provide summaries of each reference this review is written as a combined overall summary of relevant key points and discusses their implications for this study.

A3.2 Australia

A3.2.1 Background

There does not appear to have been an automatic link between the timing of higher mass limits and adoption of higher bridge design loads. Bridge design loads and mass limits have been increased at different times with the latter occurring more frequently. Table A.1 shows the history of bridge design load increases in Australia.

Table A.1   History of bridge design loading in Australia (NSW)

Era	Load description	Mass
Pre-1948	A36	Rigid truck – gross mass 15–17t – plus an additional uniformly distributed load
1948 – 1976	MS18/H20-S16	Semi-trailer – gross mass 33t
1976 – 2004	T44	Semi-trailer – gross mass 44t
2004 – present	SM1600	Combination of axle set, single vehicle and queue of vehicles

Source: Bushby (2004)

Figure A.1 shows the trend of mass limits in the State of Victoria over a similar period. It can be seen that the mass limit has increased from 25t to 42.5t, excluding combination vehicles, over the last 50 years. In fact it has been reported that the loads applied to Australian bridges have been increasing at a rate of 10% per decade since the beginning of the 1900s (Heywood et al 2000).

Figure A.1   Trend in Victorian mass limits (tonnes)
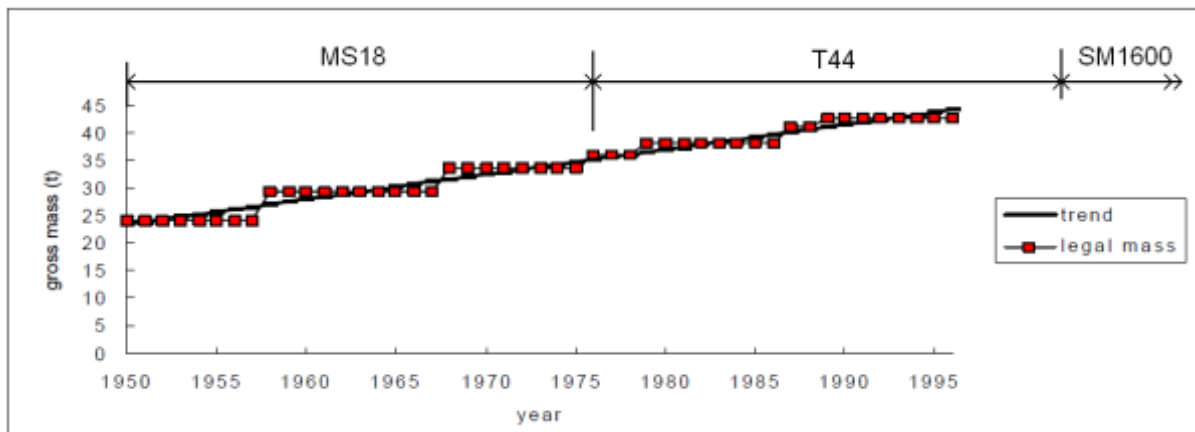
Source: Bureau of Transport and Communications Economics (1997).

Recent road freight projections by the Bureau of Infrastructure Transport and Regional Economics (BITRE) assumed that between 1999 and 2030, heavy vehicle productivity would improve by 2% per annum for inter-regional road freight, and 1% per annum for shorter-distance intra-regional road freight. BITRE noted that these assumptions were in line with trends in road freight vehicle average loads observed over the last 10–15 years and also reflected the potential future uptake of larger heavy vehicle combinations, such as B-triples and quad axle vehicles, available under performance based standards vehicle regulations.

Studies of proposed increases in heavy vehicle mass limits in Australia (NRTC 1996) and New Zealand (Travers Morgan 1995) have consistently found that the benefits of allowing heavier trucks outweigh the costs and that limits should be increased. However, the studies have almost always only considered modest increases in vehicle mass limits. During the Mass Limits Review (NRTC 1996) interest began to grow in determining whether there was a limit above which it would never be economic to provide for heavier vehicles. It was considered important to look forward to what may be desirable options, and ensure they were not ruled out, rather than being unduly concerned about constrained incremental improvements to the existing situation. This review also confirmed that bridge strength was proving to be a limit to any mass increases and productivity of the road transport system. It appeared unreasonable for bridges to be limiting productivity given the small percentage of investment in bridge infrastructure.

A3.2.2   Austroads Project RUM.H.96: Economics of higher bridge design loadings (1996–1999)

In 1996 Austroads commissioned a project specifically to investigate the economics of higher bridge design loadings (Pearson and Bayley 1997). The project was approached from a road user perspective and aimed to look well beyond previous incremental increases in legal loading to try to find an end point in the evolution of the Australian general freight vehicle. The project was not constrained by existing mass limits or bridge infrastructure but sought to determine the economic optimum, starting almost from first principles.

The project aimed to meet the following objectives:

- The design load model should provide an envelope of forces that would ensure structures were covered for:
  - all present Australian general access vehicles, including the emerging B-triples and the currently restricted triple road trains
  - present and projected special purpose vehicles

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- projected potential future general access vehicles – ideally the load model should cater for their ultimate evolution, but should aim to cover at least the next 50 years so that structures designed to this loading in 20 years time would have a minimum life of 30 years.

- The design load model should provide the designer with a simple realistic image of how traffic and hence forces could load a structure.

The project comprised the following stages:

Stage 1	Vehicle tasks	Calculate benefits arising from increased laden mass of vehicles
Stage 2	Bridge tasks	Calculate increased construction cost for new bridges for different vehicle configurations and loadings
Stage 3	Economic tasks	Determine which loading condition is the most economic
Stage 4	Standard load task	Determine a new bridge design loading for the most economic loading condition

Stage 1 is described in a working paper by Pearson and Bayley (1997). The two broad objectives for this stage were to:

- identify alternative laden vehicle mass generally achievable either within present dimension limits or through using shorter trailer combinations

- estimate reductions in transport costs from higher truck laden mass for the alternative mass and vehicle scenarios.

Vehicle payloads are determined by dimension limits and freight density which in turn varies by commodity type. Recognising that it would not be economic to provide sufficient bridge strength to handle the largest trucks loaded with the densest commodity types, the study sought to identify the density at which 90% of the freight task would be volume constrained, ie only 10% of the task would be constrained by mass limits. Even with existing mass and dimension limits some lower density freight types were already volume constrained rather than mass constrained. The threshold commodity and density determined in this way was coal with a practical density of 0.81t per cubic metre. For road safety reasons it was decided that the maximum density should be reduced to 0.73t per cubic metre as this was the density at which fully laden vehicles would have roll stability not lower than that exhibited by the loaded vehicle fleet. This maximum density covered 80% of the freight task.

The study then considered vehicle configurations and mass and adopted the following two classes of vehicles to represent the various alternatives:

- vehicles at present maximum length – L group vehicles

- shorter vehicles with better swept path performance – S group vehicles.

The L group vehicles were constrained to remain broadly within current maximum length limits. Existing dimension limits, including height and width, were seen as so intrinsic to the total road infrastructure that they were unlikely to change significantly.

Table A.2 shows the lengths of different vehicle types in the two vehicle length classes.

Table A.2   Vehicle lengths

Vehicle type	L group length (m)	S group length (m)	Criteria for determining S group lengths
Articulated vehicle	19.0	14.8	Same swept path as 12.2m rigid truck
B-double	25.0	21.4	Same swept path as 19m articulated vehicle
B-triple	33.0	27.9	Same swept path as 25m B-double
Road train triple	53.5	40.6	Based on short articulated vehicle length and two additional trailers

The study calculated the freight density corresponding to the threshold between volume and mass constrained loads at existing mass and dimension limits to be 0.28t per cubic metre. For the analysis three intermediate load densities were selected between 0.28 and 0.73t per cubic metre and two intermediate lengths S1 and S2 equally spaced between the S group and L group lengths.

The various load densities were applied to the different length vehicles and axle set and gross masses were calculated. The gross mass results were found to be much higher than the existing mass limits.

The third task in this working paper was to calculate benefits that could be achieved from future higher mass limits. This involved estimating the likely utilisation of higher mass limits and transfers between vehicle configurations. Assumptions in the calculation of utilisation included that only larger vehicles with six or more axles would move to the higher limits and also that there would be no benefit for existing volume constrained commodity types below 0.28t per cubic metre. Shifts between vehicle configurations could occur if freight operators transporting denser commodities chose to use shorter vehicles for these loads to take advantage of better swept path performance and therefore greater access, rather than putting larger loads on long vehicles.

Benefits arose because of the reduction in vehicle travel due to higher payloads. The savings from reduced travel of the fleet far outweighed the increased operating costs of the heavier vehicles.

Table A.3 shows the benefits for the L group (legal length) vehicles at higher payloads.

Table A.3   Benefits from higher payloads – L group ($million pa, resource costs, 1994/95 dollars)

Group	Nominal payload on articulated vehicle (tonnes)	Gross benefit	Relative benefit (per tonne of increase in payload on articulated vehicle)
L28 (existing)	26.5	-	-
L43	40	587	43
L51	48	789	37
L58	54	875	31
L73	70	1099	25

Table A.3 illustrates the following points:

- The benefits are very large – even in 1994/95 dollars and with the lower freight volumes being transported at that time benefits were up to $1 billion per annum.

- There are diminishing returns to further increases in payload at higher loads. This is due to the general nature of freight task. As the allowable mass gets larger, the percentage of the freight that is of sufficient density to allow vehicles to utilise this additional allowance continually diminishes.

- Further analysis indicated that benefits flatten off above 40 – 45t payload (on an articulated vehicle) and increased payload above 70t yields little incremental benefit.

Similar results and trends were found for the S group vehicle configurations.

The shorter vehicle category may be relevant to New Zealand because the original Transit NZ heavy routes study indicated that geometric limitations were by far the most expensive constraint to increased heavy vehicle access (Wanty and Sleath 1998). Nevertheless many transport operators want to be able to carry more volume-constrained freight per vehicle.

Stage 2 of the Austroads study investigated increased bridge construction costs (Austroads 1997a). A summary of the stage 2 work (Austroads 1997b) indicated that:

- Bridge construction costs are not highly sensitive to increases in design vehicle mass, increasing approximately 1% for each 10t increase in axle set loading.

- Bridge construction costs also vary with the configuration of the vehicle, thus moving from a semi-trailer as the design vehicle, to a B-double, and then to a B-triple, results in further increases of around 0.5% at each step.

- Bridge construction costs also vary with the length of the design vehicle. Freight vehicles shorter than current dimensions would be very beneficial for carrying higher density freight. The relative increase in bridge construction costs due to reduced length varies between vehicle types, from 0.5% for a semi-trailer to 1% for a B-triple.

In the stage 1 working paper Pearson noted that the T44 bridge design load represented a 25% increase over the previous design load but only increased bridge construction costs by about 5%.

Corresponding percentages would need to be estimated or verified for New Zealand and applied to annual bridge construction expenditure to determine the ratio of benefit to cost.

Other working papers considered vehicle stability, safety and driveability of the potential higher mass vehicles, and the cost of increased pavement wear.

Both current and potential technological changes in vehicle design were assessed for a range of operational criteria including lateral stability, braking, ability to maintain speed on grades and high-speed dynamic off tracking.

The pavement assessment found that pavement wear was not a compelling reason to prevent increased axle set mass until at least the 30–35t level but would be expected to rule out increases beyond the 40t level.

Based on the above assessments it was concluded that a suitable envelope upon which to base the bridge design loading model would be the L58 long and S73 short vehicle groups as detailed in table A.4.

Table A.4   Maximum likely feasible future vehicle configurations and mass

Group title	Vehicle	Length (m)	GVM (tonnes)	Axle set loadings		
				Steer	Dual	Tri-axle
L58 (freight density 0.58t/cu m.)	Articulated	19.0	74	7	27	40
	B-double	25.0	114	7	27	40
	B-triple	33.0	154	7	27	40
	Road train	53.5	208	7	27	40

Group title	Vehicle	Length (m)	GVM (tonnes)	Axle set loadings		
				Steer	Dual	Tri-axle
S73 (freight density 0.73t/cu m.)	Articulated	14.8	73	7	26.5	39.5
	B-double	21.4	112.5	7	26.5	39.5
	B-triple	27.9	152	7	26.5	39.5
	Road train	40.6	205	7	26.5	39.5

Note: GVM = gross vehicle mass

A separate paper noted that another major issue with increasing the design loading was whether the anticipated higher mass limits could feasibly be accommodated on the road system, or whether the capital expenditure required to upgrade the existing infrastructure would preclude this eventuality (Gordon and Boully 1998). The paper made a brief assessment of this issue, not proposing such an increase, but assessing if it could be justified in the future. The main finding of this assessment was that a national project to boost the strength of bridges to provide for a minimum of a 30t axle set loading capacity would be both affordable within current budget levels, and also highly viable. It would have a benefit–cost ratio comparable to or better than existing major road infrastructure projects such as freeway construction.

The foregoing summary describes how the traffic loading envelope was developed for the current bridge design load in Australia based on long-term economic considerations. It has been described in some detail so that the reader can understand where the approach and findings might be applicable in New Zealand and conversely where differences in traffic density, terrain, existing infrastructure standards and policy settings, etc might lead to different results for New Zealand.

A3.3 New Zealand

A3.3.1 Background

Following the adoption of the HN-HO-72 bridge design load in 1972 there was a major bridge replacement programme through the 1970s and 1980s. Until 1989, roads in New Zealand were classified into Class I and Class II with lower weight limits applying to Class II roads. Most rural local authority roads were Class II to protect pavements and old bridges but there were difficulties in monitoring heavy vehicle compliance with the different weight limits. There was also pressure from the transport industry to remove the distinction and increase mass limits across the board. A thorough review of the structural capacity of the New Zealand road network resulted in the adoption of the 44t gross weight limit and all Class II roads were reclassified as Class I from 1 February 1989. However axle weight limits remained below those of many other countries to minimise pavement wear.

A3.3.2 Transit New Zealand Heavy Vehicle Limits Project (1994–2001)

Within five years, the road transport industry again began seeking higher mass limits, at least on particular routes. In response Transit NZ began an investigation into the feasibility of heavy transport routes. Transit NZ completed a final report on this Heavy Vehicle Limits Project in 2001.

A 1995 conference paper (Sleath 1995) describes how an industry survey was undertaken to identify heavy traffic flows in terms of both vehicle flows and tonnage, and to obtain other information such as the level of interest in an increase in the legal axle limit. Survey questionnaires were mailed out to 1380 transport operators and customers. The total response was 44% of those mailed, of which 489 returns were complete and used in the analysis. The respondents covered a wide range of operators and freight customers.

Responses to the question about interest in an increased legal axle limit were analysed for each route and respondent, and by whether the route was on state highways or local authority roads. Results are shown in table A.5.

Table A.5   Interest in increased axle limit

Type of road	Advantage seen	%	No advantage seen	%	Number of routes
State highway	1293	61	823	39	2116
Local authority	356	46	420	54	776
Total	1649	57	1243	43	2892

The overall response showed a perceived advantage from increasing axle limits in 57% of the cases. However, for local roads, less than half the routes identified were seen as benefiting from heavier axle limits. These results indicate that the transport industry does not see an automatic need for increased weight limits across all roads. This may reflect a short-term outlook based on their existing vehicle fleets and freight task but also may be a realistic assessment reflecting awareness that any higher costs would need to be fully recovered from the industry through higher road user charges (RUC).

A pilot route from a forestry and dairying region to an export port was selected for detailed evaluation. Evaluation of the pilot route was made using eight vehicle types, distinguished by axle configuration and comprising six, seven and eight axle vehicles.

Seven of these vehicles were selected as representing typical examples of potential configurations and were all within (then) current vehicle dimension limits. In addition a 27m A-train logging vehicle was tested to show the effect of varying the vehicle length.

Transit NZ's HPermit software was adapted to assess the capacity of bridges to carry heavier vehicles. The study identified the limiting bridge along the route for each vehicle type and compared its capacity with the current legal weight limits. It was found that bridges on the pilot route could accept significant increases in the current legal gross vehicle weights for the vehicle types considered. The increases ranged from 12% to 25% for the different vehicle types. Some of the vehicle types would also require increases in legal axle weights of up to 20% in order to utilise such higher gross weight limits.

A second paper (Wanty and Sleath 1998) reported on the second stage of the study. It notes that the original purpose of stage 2 was to evaluate the economic benefits of developing some heavy transport routes, based upon the results of stage 1. When embarking on stage 2 Transit NZ was concerned that there would be practical difficulties in reintroducing a road hierarchy based on different weight limits, as had existed prior to 1989. Hence it was decided to examine both the concept of designated heavy transport routes and an overall increase in legal weight limits applying to all roads.

From a comparison with other countries it was concluded that there was a significant range of gross vehicle weight limits between countries for equivalent classes of heavy vehicles, and that New Zealand tended to be at the lower end for both these and axle weights. It could be inferred from this that there were potential net benefits to be gained in New Zealand by raising the legal load limit. New Zealand's dimension limits were comparable with those in the other countries. This appeared to indicate that dimension limits would not be a controlling factor.

From discussions which took into consideration bridge loading effects, the viewpoint of the New Zealand road transport industry and overseas practice, three load limit options were chosen for evaluation. When expressed as ratios of New Zealand's highway and bridge design load standard (known as HN) these were equivalent to 0.93HN, 1.00HN and 1.15HN. These options correspond to axle and group load limits shown in table A.6.

Table A.6   Existing and proposed axle and group load limits (tonnes)

Axle type	Wheelbase	Current (0.85HN)	0.93HN	1.00HN	1.15HN
Twin tyred axle		8.2	9.0	9.0	9.0
Tandem axle set	1.0–2.0m	14.5–15.5	15.4–16.4	16.0–17.0	16.5–17.5
Tri axle set	2.0–3.0m	15.5–18.0	17.3–19.1	18.6–20.5	20.5–22.9

Note: Table A.6 is a summarised version of the original.

The options result in the gross load limits shown in table A.7.

Table A.7   Existing and proposed gross load limits (tonnes)

Vehicle type	Axles	Current	0.93HN	1.00HN	1.15HN
Truck/trailer	8	42.0	52.0	55.8	62.0
	9	42.0	53.7	57.6	65.6
Semi-trailer	6	39.0	40.3	42.3	45.0
A-train	6	39.0	48.9	49.5	50.0
B-train	8	44.0	55.9	58.8	62.0
	9	44.0	55.3	59.3	67.0

Note: Only a representative selection of vehicle/axle number combinations is shown.

Existing width and height limits on vehicles were not changed. However, lengths were increased to match the chosen gross vehicle weights.

The benefits of an increase in load limits arise from increased productivity associated with the transport of larger loads. These gains are measured as a reduction in freight charges per tonne-km. The gains are partially offset by the additional costs of operating vehicles at higher weights.

The evaluation of total freight tonne-km was made using RUC records of weight and distance, the National Traffic Database, and road transport industry information on vehicle and commodity types.

Uptake to heavier vehicles was determined in consultation with the New Zealand Road Transport Association. It was influenced by the type of commodity involved and by the extent to which the load increase is determined by volume or weight.

To estimate the net productivity benefits, vehicle freight rates were then determined from vehicle operating cost models for existing and new vehicles. A significant factor here was the assumed distance travelled each year. In consultation with the transport industry this was finally set at an average value of 80,000km/year. Vehicle benefits were then calculated by determining total freight transport costs with and without the change to heavier vehicles.

Costs were estimated for additional pavement rehabilitation and bridge strengthening and also for upgrading sharp bends, narrow road lanes, and roundabouts to handle the greater swept paths of the proposed longer vehicles. No evaluation was made of net costs or benefits of safety issues – this was to be considered once the economic viability of higher weights had been demonstrated.

The economic evaluation results are shown in table A.8.

Table A.8   Economic evaluation results

	Load limit		
	0.93HN	**1.00HN**	**1.15HN**
Costs – present value			
Pavements ($M)	161	208	252
Bridges ($M)	32	48	82
Corner widening ($M)	1719 (142)	1719 (142)	1719 (142)
Total costs ($M)	1912 (335)	1975 (398)	2053 (476)
Benefits – present value			
Total benefits ($M)	322	384	450
Net present value ($M)	-1590 (-13)	-1591 (-14)	-1603 (-26)
Benefit–cost ratio	0.17 (0.96)	0.19 (0.96)	0.22 (0.95)

Note: The figures in brackets relate to costs excluding roads in mountainous terrain.

On the basis of these results it was concluded that a general increase in mass limits and vehicle length to accommodate vehicle types of up to 65t gross and 26m in length was uneconomic, being penalised particularly by the high costs of corner widening. A general increase in mass limits was marginally economic if corner widening was excluded on roads in mountainous terrain.

Some observations on these results are:

- The benefits are much less than those obtained in the Australian studies – note that these are present values for 30 years of benefits whereas the Australian benefits were per annum. Reasons for this are likely to include longer freight haul distances and less mountainous terrain in Australia, a smaller freight task and fewer trucks in New Zealand and a smaller proposed increase in vehicle loads.

- The pavement and bridge costs are overwhelmed by the corner widening – it seems unrealistic to load the project with this cost and not attribute any safety and other benefits for general traffic. It is notable that when roads in mountainous terrain are excluded the benefits almost equal the costs.

- Bridge strengthening costs and the proportion of bridges requiring strengthening are small, even to accommodate the highest loads equivalent to 1.15HN. This implies that there is considerable spare capacity in existing bridges without even increasing bridge design loads. It would also appear, based on the Australian studies, that the incremental cost of building bridges for higher design loads likely to be justified in New Zealand would be very small.

In the light of these results Transit NZ commissioned a heavy vehicle tracking project and focused further investigations on the following two scenarios:

- Scenario A – Operating the existing vehicle fleet at higher weights (hence avoiding corner widening costs).

- Scenario B – Increasing vehicle dimensions and weights on upgraded selected routes (key freight routes that already have a good standard of geometry).

This work was completed in 2001 and found that both of the additional scenarios were economic, with scenario A offering considerably greater economic benefits than scenario B. However, the findings were not implemented. In response to further persuasion from industry the Ministry of Transport and the NZ Transport Agency (the Transport Agency) recommended investigations in 2006 and this culminated in the VDAM Rule Amendment 2010 providing for HPMV routes.

A3.3.3   Land Transport Rule: Vehicle Dimensions and Mass Amendment 2010 (2006–2010)

The following information on the economic benefits likely to result from the VDAM Rule Amendment 2010 is obtained from the Ministry of Transport's (2010) *Regulatory impact statement* and the Pearson (2007) report commissioned by the Ministry of Transport and Transit New Zealand.

The original dimension and mass rule amendment proposal included an option of an across-the-board increase in mass and dimensions (as envisaged in scenario A of Transit NZ's Heavy Vehicle Limits Project). An across-the-board (or as-of-right) increase to 50t from the current 44t gross mass limit and an increase in permitted vehicle length to 22m from 20m were considered. The bridge and pavement studies assessed this option and it was found to be economically justified. However in the formal consultation concerns were expressed that there could be adverse effects on road infrastructure, pavements and especially bridges, with the potential for catastrophic outcomes. This was of particular concern to road controlling authorities who preferred a controlled permit system that allows for route assessments to ensure they are appropriate to the heavier vehicles. This would not be possible with an across-the-board increase.

The Ministry of Transport initiated heavier vehicle trials throughout 2008 and early 2009 based on increasing the gross mass of some existing 20m vehicles from 44t to around 50t. Such an increase was found to:

*   increase productivity by approximately 16%

*   reduce the number of trips by around 16%

*   reduce fuel use by about 20%.

The above productivity benefits (defined as a reduction in the number of trips to move the same amount of tonnage) are net of the substantial additional RUC that will accrue with the increases in mass. Heavier vehicles incur substantially higher RUCs through the operation of the scale of charges that relates RUC to the 4th power rule, which in turn reflects increased pavement wear. Essentially this means that transport operators meet any additional pavement maintenance costs over time.

In the above trial the RUC increased from $0.3513 per km at 44t to $0.522 per km at 50t (a 48% increase in RUC for a 14% increase in load). This increase was partially offset by the reduction in total kilometres to complete the same freight task but the total RUC was still 25% higher using vehicles loaded to 50t. As noted above the 16% productivity benefit is net of this increased RUC.

The main project methodology for the Pearson report was the preparation and circulation of questionnaires to major providers of transport services and users of transport services. The questionnaire sought information on potential routes and vehicle mass and dimension concessions that respondents would be interested in. The proposals were assessed and ranked on their alignment with the objectives of the New Zealand Transport Strategy.

Based on an analysis of potential concessions, the report concluded there would be a:

*   16% reduction in annual distribution costs

*   20% reduction in road trips

*   16% reduction in fuel usage

*   reduction of between 5% and 10% in operator transport costs

*   10% reduction in inventory at the end of the distribution chain.

The Pearson Report also concluded that transport costs could be reduced by between $100 million and $200 million annually (how this reduction is shared between operators and users depends on commercial

considerations). The study also calculated cost–benefit ratios for three companies which were calculated at 17, 13.2, and 12.7 if they were to operate at around 50t on selected routes.

The trials carried out by the Ministry of Transport found similar results to those of the Pearson Report in terms of productivity increases, trip and fuel reductions.

Earlier in 2010, at the request of the Ministry of Transport, the Transport Agency undertook an assessment of the bridges on the state highway network that were most likely to be used for heavy vehicle movements. The study found that 306 state highway bridges required work, and the estimated cost of strengthening and replacing bridges from this study is $85 million (with a best case/worst case scenario range of $60 to $190 million). This is a relatively small one-off cost relative to the potential annual benefits identified above.

A report on an evaluation of the first year of operation of the VDAM Rule Amendment 2010 (Stimpson and Co 2011) noted that the Transport Agency asset manager had commissioned research on HPMV cost impacts on structures. Early conclusions from this work were that cost impacts on structures were likely to be low. However, the report also found that almost half of higher mass permit applications were declined, primarily because of bridge limitations. From this it appears that even if the cost impacts on structures are low there are existing constraints that are limiting the realisation of some of the potential economic benefits from HPMVs.

A3.3.4 Optimisation of heavy vehicle performance

In *NZ Transport Agency research report 387* (Taramoeroa and de Pont 2007) the performance of typical vehicles used in six transport tasks in New Zealand was benchmarked against that of vehicles undertaking those same tasks in Australia, Canada, Southeast Asia and the UK. The six transport tasks analysed were passenger coach transport, bulk liquids and bulk materials transport, 40-foot ISO intermodal container transport, and livestock and refrigerated goods transport. A demonstration of a more optimal New Zealand truck and full trailer was presented, and ways to optimise other vehicle configurations were discussed. However, while small increases in vehicle loads were proposed, these increases were incremental and hence not as useful for projecting future high productivity vehicle types and loads in New Zealand as expected.

A3.3.5 Economic evaluation manual, volume 1, January 2010

Relevant provisions from the EEM are:

- The discount rate is 8% per annum. The manual indicates that lower discount rates of 4% and 6% can be used for evaluations of activities that have long-term future benefits that cannot be adequately captured with the standard discount rate. This would apply to long-life assets such as bridges.

- The analysis period is 30 years. A 30-year analysis period would appear too short to really capture the benefits of a higher bridge design load enabling higher mass limits in the longer term. It is suggested that if some form of economic analysis is considered necessary for this project it should consider a longer than standard analysis period.

- The EEM states that if a bridge serves little traffic and is expensive to replace, a replacement option should not automatically be taken as the do-minimum, particularly if alternative routes are available to traffic presently using the bridge. In this case the do-minimum may be to not replace the existing bridge and to have no bridge. This suggests that economic assessments of as-of-right higher mass limits should not necessarily assume that all bridges on low-volume roads need to be considered.

A3.3.6   Other issues and discussion

Other factors to be considered in the development of a new bridge design loading for New Zealand are as follows.

Further investigation is required into limitations on increased vehicle loading in New Zealand. There is likely to be a range of factors that are specific to New Zealand that will limit the applicability of vehicle types and loadings common in other countries. The most significant is possibly the RUC system. The existing scale of charges rises steeply for heavier vehicles especially those with heavier axle loads. In other countries road cost recovery regimes often tend to under-recover from large heavy vehicles such as B-doubles and over-recover from smaller heavy vehicles. Existing RUCs may mean that the opposite is the case in New Zealand and this may diminish the attractiveness to transport operators of moving to the much heavier trucks operated in other countries. Consideration may need to be given to the likelihood of any changes to the basis of heavy vehicle charging and to reviewing axle designs of vehicles to manage RUCs better by increasing the number of axles of heavy vehicles and utilising tri and quad axles. Steerable axles will also help overcome corner widening costs. The investment in corner widening will also come under pressure from road safety and increasing traffic volumes and this should be taken into consideration in assessing the overall economic benefits.

Lower heavy vehicle traffic volumes suggest that the optimal upper limit of vehicle loads is lower than in countries with much higher heavy vehicle volumes, at least for as-of-right higher limits on all roads. It is arguable that because the incremental costs of building stronger bridges are so small this should not be an issue; however, this should be assessed rather than assumed.

If New Zealand has a significant legacy of existing bridges that were only renewed in the last 30 to 40 years and hence are not due for replacement for some time it could be decades before any significant additional strength built into new bridges today could be utilised, at least by across-the-board mass limit increases. This would reduce the economic case for a significantly higher bridge design loading in present value terms.

Before the recent VDAM Rule Amendment 2010, New Zealand's mass limits had not changed for 20 years. Any trends observed from WiM data over the latter part of this period were likely to be flatter than if there had been more frequent increases in mass limits. Also in the longer term, at much higher loads, the Australian investigation found that the uptake of further increases in mass limits was likely to decrease. These factors mean that extrapolating from recent WIM data is unlikely to be a sound basis on its own for deciding an appropriate new bridge design loading.

There have already been over 1000 applications for HPMV permits under the VDAM Rule Amendment 2010. This indicates there is a large demand for higher mass limits. The applications could also provide a valuable indicator of the direction in which vehicle types and mass limit increases might go and hence should be considered for future bridge design loadings.

A4   New Zealand

A4.1   NZ Transport Agency *Bridge manual*

A4.1.1   General

The *Bridge manual* was first published in May 1994 and the second edition was published in June 2003, with subsequent amendments to sections in June 2004, September 2004, December 2004 and July 2005. The third edition (NZ Transport Agency 2013) is reviewed below.

The *Bridge manual* sets out the criteria for design of new bridges and evaluation of existing structures for road and/or pedestrian traffic with main members spanning up to 100m.

A4.1.2 Traffic load

Section 3.2 of the *Bridge manual* specifies traffic loads – gravity effects, and section 3.3 specifies traffic loads – horizontal effects.

The specified traffic loading elements consist of the HN-HO-72 loading model which includes an element to represent the normal loading (HN) and an element to represent an overload loading (HO). The loading elements are shown diagrammatically in figures A.2 and A.3 below.
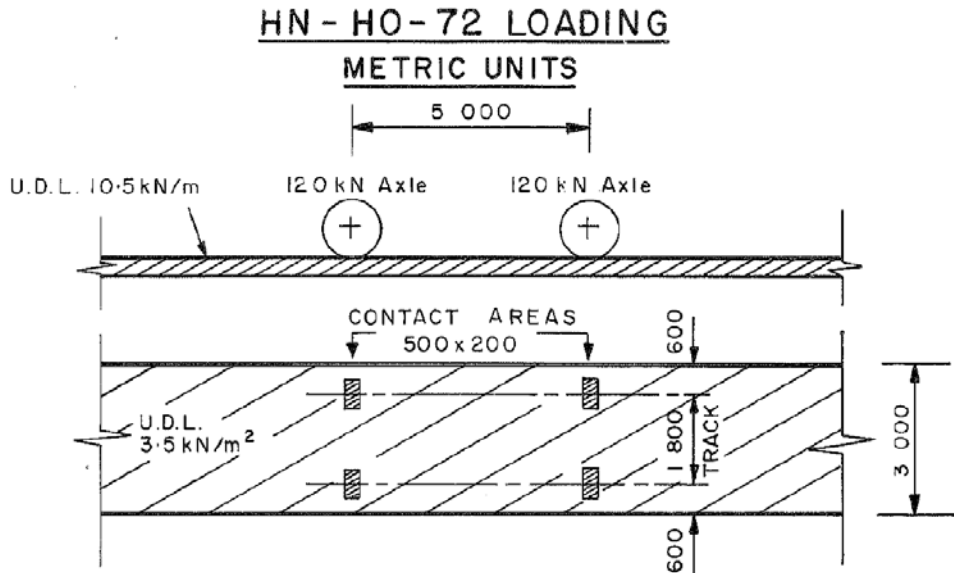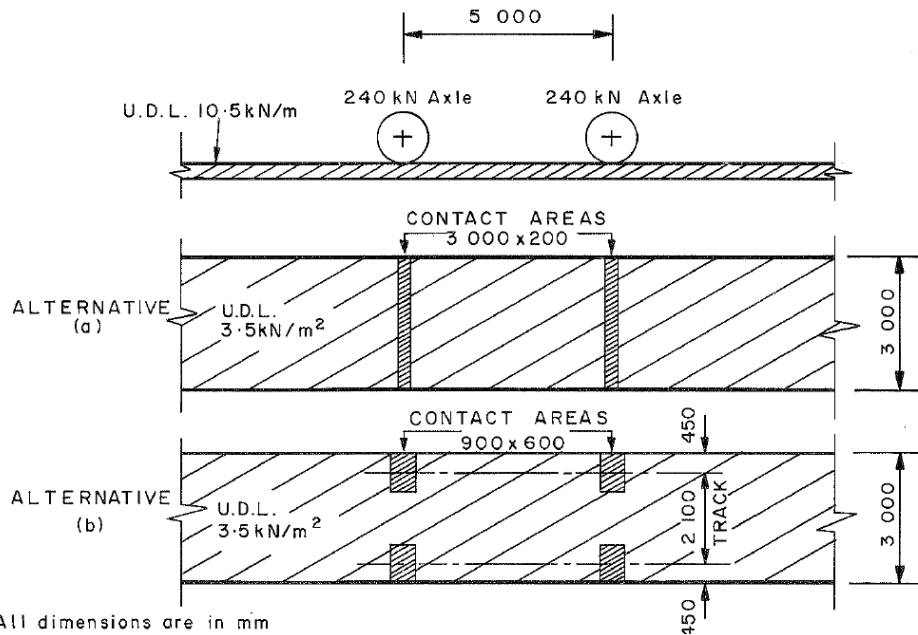
Figure A.2   HN loading element

Figure A.3   HO loading element

The HN loading element includes the following:

• a 3.5kPa uniformly distributed lane load which equates to a 10.5kN/m lane load for a 3m wide lane

- 2 x 120kN vehicle axle loads which equate to a maximum vehicle weight of 240kN

- a fixed axle spacing of 5000mm and a transverse wheel spacing of 1800mm

- the number of HN loading elements is a maximum of one element per lane and as many lanes loaded to give the most adverse effects.

The HO loading element includes the following:

- a 3.5kPa uniformly distributed lane load which equates to a 10.5kN/m lane load for a 3m wide lane

- 2 x 240kN vehicle axle loads which equate to a maximum vehicle weight of 480kN

- a fixed axle spacing of 5000mm and a transverse wheel spacing of 2100mm

- the number of HO loading elements is a maximum of one element per bridge and replaces any one HN loading element in the combination to give the most adverse effects.

The wheel loading element for the design of local effects is the maximum of the following to give the most adverse effects:

- 60kN over a wheel contact area of 500mm x 200m (600kPa)

- 120kN over a wheel contact area of 900mm x 600mm (222kPa)

- 240kN over a wheel contact area of 3000mm x 200mm (400kPa).

A4.1.3 Lane widths and number of lanes

The design lane width for the traffic loading is specified as 3m wide.

The number of design lanes allowable for the width of a bridge is determined from the tabulated range shown in table A.9.

Table A.9   Number of load lanes

Width of roadway	Number of load lanes
Less than 6.0m	1
6.0m but less than 9.7m	2
9.7m but less than 13.4m	3
13.4m but less than 17.1m	4
17.1m but less than 20.8m	5

A4.1.4 Lane modification factors

The lane modification factors for a multiple lane loading are shown in table A.10.

Table A.10   Lane modification factors

Number of lanes	Modification factor
1	1.0
2	0.9
3	0.8
4	0.7
5	0.6
6 or more	0.55

The modification factor applies to all lanes concurrently.

A4.1.5   Dynamic load factors

The dynamic load factor (DLA) is related to the span of the bridge and the location of the design element in consideration.

For reaction and shear, and for bending in deck slabs and cantilevers, the DLA is 1.30.

For bending in other elements above ground the DLA is 1.30 up to a span length of 12m. For spans greater than 12m the DLA varies depending on the span length and is defined by the equation below:

$$Dynamic\ Load\ Factor\ (I) =\ 1.0 + \left[\frac{15}{(L + 38)}\right]$$

where:

L is the span length (m).

This results in a dynamic factor of 1.20 for a 37m span and 1.10 for a 112m span.

A4.1.6   Horizontal loads

Braking forces

For local effects the braking load is defined as 70% of a HN axle load which equates to an 84kN load.

For global effects the braking load is defined as the greater of either 70% of both HN axles which is a 168kN load or 10% of the total live load.

The braking force is applied longitudinally and at the deck surface level.

Centrifugal force

The centrifugal force is defined as a proportion, C, of the live load and determined by the design speed and radius of the road and is determined by the following equation:

$$C = \frac{0.008S^2}{R}$$

Where:

S is the design speed (km/h) and R is the radius (m).

The centrifugal force is applied as a radial force and at 2m above the road surface level.

A4.1.7   Load factors

The live load factor for the SLS is currently specified as 1.35.

This serviceability live load factor was updated from 1.0 to 1.35 following a review of load factors carried out in 2002.

The ratio of ultimate live load to SLS live load is currently specified as 1.67 which is multiplied by an overall factor of 1.35 to provide a total load factor on a nominal live load of 2.25.

Tables A.11 and A.12 show combinations of loads and load factors.

Table A.11   Load combinations – serviceability limit state

Load	Load factors				
	Combination 1A	Combination 2A	Combination 2B	Combination 2C	Combination 4
Dead load	1.0	1.0	1.0	1.0	1.0
Locked in forces	1.0	1.0	1.0	1.0	1.0
Ground water	1.0	1.0	1.0	1.0	1.0
Earth pressure	1.0	1.0	1.0	1.0	1.0
Ordinary water pressure	1.0	1.0	1.0		1.0
Shortening effects	1.0	1.0	1.0	1.0	1.0
Settlement	1.0	1.0	1.0	1.0	1.0
Centrifugal effects	1.0	1.0	1.0	1.0	
Live load (HN)	1.35	1.35	1.35	1.35	
Overload live load (HO)					1.0
Pedestrian & cycle load	1.0	1.0	1.0	1.0	0.5
Temperature effects		1.0			0.5
Horizontal effects (braking and centrifugal)		1.0	1.0	1.0	
Wind load			1.0		
Flood water pressure				1.0	
Water ponding				1.0	

Table A.12   Load combinations – ultimate limit state

Load	Load factors				
	Combination 1A	Combination 2A	Combination 2B	Combination 2C	Combination 4
Dead load	1.35	1.2	1.35	1.35	1.35
Locked in forces	1.35	1.2	1.35	1.35	1.35
Ground water	1.0	1.0	1.0	1.0	1.0
Earth pressure	1.82	1.2	1.35	1.35	1.35
Ordinary water pressure	1.35	1.2	1.35		1.35
Shortening effects	1.35	1.2	1.35	1.35	1.35
Settlement	1.35	1.2	1.35	1.35	1.35
Centrifugal effects	2.25	1.2	1.35	1.35	1.49
Live load (HN)	2.25	**1.2***	**1.35***	**1.35***	
Overload live load (HO)					1.49
Pedestrian & cycle load	1.75	1.2	1.35	1.35	0.7
Temperature effects		1.2			0.33
Horizontal effects (braking and centrifugal)		1.2	1.35	1.35	
Wind load			1.0		
Flood water pressure				1.0	
Water ponding				1.0	

* Note: The ULS load factors for the live load for the load combinations 2A, 2B and 2C are less than the proposed SLS load factors for the same combinations.

A4.1.8   Evaluation

Bridges in New Zealand are evaluated to obtain parameters which define their load carrying capacity. Evaluation is carried out for three loading levels and two parameters are determined for each evaluation level: one for the main members and one for the deck. These evaluation levels are:

- rating evaluation – defines the capacity of bridges to withstand overweight vehicles

- posting evaluation – defines the capacity of bridges to withstand Class 1 vehicles

- HPMV evaluation – defines the capacity of bridges to withstand HPMV vehicles.

The rating evaluation determines the two required parameters as a percentage for main members and as an alphabetic grade for decks derived from deck capacity factors. The posting evaluation determines the two required parameters as a percentage for main members and as a specific axle load for decks. The HPMV evaluation follows the same procedure for the posting evaluation but the evaluation is based on the HPMV loading vehicle.

The rating load is defined as: one lane containing an overweight vehicle (0.85HO), plus other marked lanes on the bridge containing the HPMV evaluation load.

The posting load is defined as: a load consisting of Class 1 conforming vehicles (0.85HN) in some or all marked lanes on the bridge.

The HPMV evaluation load is defined as: HPMV conforming vehicles in some or all marked lanes on the bridge. The HPMV conforming vehicle loading is 0.90HN loading for spans up to 25m and 0.95HN loading for spans greater than 25m.

Evaluation of main members

The evaluation procedure involves determining the residual capacity of an element to withstand a specified live load. This is determined by subtracting the dead load and other permanent or transitory effects from the capacity of the element to give the remaining or residual capacity of the element that is available to withstand live load effects. This residual capacity is then divided by the load effects of a specified live load vehicle which gives an indication of the capacity of elements to withstand that particular loading.

The following equations are used to evaluate the rating and posting at the ULS.

Rating load evaluation:

$$CLASS = \frac{\emptyset R_i - \gamma_D(DL) - \sum(\gamma(Other\ Effects))}{\gamma_o(Rating\ Load\ Effect)} \ x\ 100\ \%$$

Posting load evaluation:

$$GROSS = \frac{\emptyset R_i - \gamma_D(DL) - \sum(\gamma(Other\ Effects))}{\gamma_L(Posting\ Load\ Effect)} \ x\ 100\ \%$$

where:

$\emptyset R_i$ is the strength capacity of the element

γ is the applicable load factor for dead load, other effects, rating load or posting load (as appropriate)

DL is the dead load effect

other effects are the other design effects in the load combination

rating or posting load effects is the live load effect for the nominated vehicles

The following equations are used to evaluate the rating and posting of prestressed concrete members at the SLS.

Rating load evaluation:

$$CLASS = \frac{Gross\ Capacity\ at\ stress\ f_o - (DL) - \sum(Other\ Effects)}{(Rating\ Load\ Effect)} \ x\ 100\ \%$$

Posting load evaluation:

$$GROSS = \frac{Gross\ Capacity\ at\ stress\ f_L - (DL) - \sum(Other\ Effects)}{(Rating\ Load\ Effect)} \ x\ 100\%$$

where:

f_o is the allowable stress appropriate to overweight vehicles

f_L is the allowable stress appropriate to conforming vehicles

Evaluation of deck members

The evaluation of deck members is either determined by an empirical method based on membrane action or by an elastic plate bending analysis.

For evaluation using the empirical method, the capacity of the deck to withstand wheel loads is dependent on deck thickness, composite action, percentage of reinforcement and concrete strength. The following equations are used to determine the rating and posting parameters and are expressed in terms of deck capacity factors for rating and allowable axle loads for posting:

For rating of deck members:

$$Deck\ Capacity\ Factor\ (DCF) = \frac{\emptyset R_i}{\gamma_o \times 95 \times I}$$

For posting of deck members:

$$Allowable\ axle\ load\ (kg) = \frac{\emptyset \times (0.6R_i)}{\gamma_o \times 40 \times I} \times 8200$$

where:

I is the DLA

For evaluation with an elastic plate bending analysis, the capacity of the deck is calculated similarly to main members and taking a design action per width of slab approach at critical sections. However the parameters are expressed in terms of deck capacity factors for rating and allowable axle loads for posting:

To calculate the rating load evaluation parameter the following equation is used.

$$Deck\ Capacity\ Factor = \frac{\emptyset\emptyset R_i - \gamma_D(DL) - \sum(\gamma(Other\ Effects))}{\gamma_o(Rating\ Load\ Effect)}$$

To calculate the posting load evaluation parameter the following equation is used.

$$Allowable\ axle\ load\ (kg) = \frac{\emptyset\emptyset R_i - \gamma_D(DL) - \sum(\gamma(Other\ Effects))}{\gamma_L(Posting\ Load\ Effect)} \ x\ 8200$$

The grade rating of the deck is then derived from the deck capacity factors.

A4.1.9 Background

The HN-HO-72 traffic live loading was first introduced in the *Highway bridge design brief (*MOW 1972) and was a development from the AASHO H20-S16-T16 design traffic loading to address concerns of possible fully laden short wheel base legal loadings exceeding the then design loading. The proposed HN loading

was based on the HA loading in BS 153 (Chapman et al 1974). The HO-72 design element was considered by the Ministry of Works and Development at the time to adequately represent the effects of one unsupervised overweight vehicle travelling in a stream of traffic.

Since the introduction of the *Bridge manual* in 1994 no changes to the HN-HO-72 design load have occurred other than revisions to the multiple lane modification factors and the SLS load factors, as mentioned above.

Since this time, the Transport Agency has commissioned a number of studies to review the adequacy of the design live load model, including:

- *Bridge manual load factors update* (Opus International Consultants 2001).

- *NZ Transport Agency research report 361*: 'Review of Australian standard AS5100 bridge design with a view to adoption' (Kirkcaldie and Wood 2008).

- *Vehicle Dimensions and Mass Rule change: development of bridge design loading for HPMVs* (Opus International Consultants 2011).

Comments on these reports are provided in the next sections.

A4.2   Bridge manual load factors update

This Opus (2001) report reviews the traffic loading of the *Bridge manual* against other international bridge standards and against actual New Zealand highway vehicle loads based on WiM vehicle load data. The WiM data includes collected data from the year of 2000 and is limited to 37 days of data or 58,000 vehicles. The methodology adopted for analysing the load effects for WiM data comprised consideration of six cases of bending moments and shear forces for single spans and twin continuous spans, for a range of span lengths.

The review included loading specifications from the UK, USA, Australia, Canada and Europe and was limited to the design traffic loading vehicles, lane loading, lane widths, DLAs and the methodology for deriving the traffic loading specification.

The following conclusions are made in the report:

- At the ULS, existing safety indices (beta factors) were found to be adequate.

- At the SLS, the existing load model was found to be inadequate when compared with the WiM data. The existing model was found to be insufficient by 20% to 35%.

The report describes the following implications as a result of the insufficient live loading model compared with the WiM data for the SLS as follows:

> Bridge elements designed at the serviceability limit state, such as prestressed concrete elements, are currently being under designed. This could mean that stresses occurring in prestressed elements are greater than they should be, with implications for crack widths and durability of prestressed elements.

> Serviceability criteria that are based on the loading model, such as vibration and deflection, are also being under designed. Again, it is expected that the criteria laid down in the design codes for these aspects of the bridge performance could be exceeded with the actual loading that is occurring on bridges.

The following recommendations are made in the report:

- Increase the uniform loading of the HN live load from 3.5kPa (10.5kN/m) to 5kPa (15kN/m). This is equivalent to a 42% increase.

- Increase the axle loads of the HN live load from 120kN to 150kN. This is equivalent to a 25% increase.

- Reduce the ULS load factor on the nominal load for combination 1 from 2.25 to 1.7. This is equivalent to a 25% decrease.

- Maintain the method of determination of the DLA; however, change the terminology of 'impact factor' to 'dynamic load factor'.

- Discard the overload HO live load for the design of main support members of new bridges. Retain the overload HO live load for the design of secondary members and for rating the assessment of existing bridges.

The loading codes reviewed in the Opus report have all since been superseded by later revisions or by new loading specifications. A later review of international loading codes by the researchers is presented in this report. The WiM data used in the Opus report is limited to a relatively small amount of data, which is not representative of the current traffic loading. The report recommends increasing the magnitude of the traffic loading model to give a closer representation to the results of the WiM data. However, the assessment of continuous bridges is limited to a smaller volume of WiM data. Also, the graphs in the report, which relate to factoring up the existing traffic loading model, sometimes differ from the results of the WiM data. This suggests that the arrangement of the existing traffic model may need to be revised to achieve a more accurate representation of the design effects of traffic loads for various bridge spans and configurations. The report also states that the WiM data was cleaned prior to using the data and the report states:

> ...records reporting vehicle speed greater than 150 kilometres per hour were removed (these records tended to describe vehicles that were excessively heavy as well as being excessively fast).

Apart from a deviation to the average results, the inaccuracy of these results is not clarified and it is not discussed whether these excessively fast and heavy vehicles may or may not exist.

The report also states that data beyond a standard deviation of +2.5 is excluded from the analysis. The report states that

> ...values greater than 2.5 standard deviations represent the highest 0.6% of the total population... The 0.6% of events from the top end of the shear and moment distribution that are excluded in the inverse-normal estimate of the ULE include events with: multiple vehicle events (for longer spans), overloaded vehicles.

This data in the highest 0.6%, if reliable, is considered to be of upmost importance when assessing the maximum actual design effects.

A4.3 NZ Transport Agency research report 361: Review of Australian standard AS5100 bridge design with a view to adoption

This report by Kirkcaldie and Wood (2008) investigates the suitability and practicality of adopting the Australian standard AS5100 (2004) for bridge design for New Zealand. The report describes the differences between the *Bridge manual* and the AS5100 and also includes recommended modifications, or additional measures required for each section of the standard to enable the potential adoption.

The report concludes that:

- for spans greater than 20m and less than 100m the M1600 moving load model is approximately twice the design loading of the *Bridge manual*

- the A160 axle loading is 1.33 times greater than that given in the *Bridge manual*

- adoption of the live loading would have significant implications for the construction cost of new bridges

- the SM1600 live load models contain variable axle set spacing and is unnecessarily complicated design loading adding to the modelling and analysis effort in the design

- the *Bridge man*ual HN and HO loadings are much simpler and give a satisfactory representation of traffic load effects on New Zealand bridges.

The Opus (2001) report described in section A4.2 states that the HN and HO loadings are exceeded regularly by the current traffic load effects on bridges. A large range of analysis and design software is currently available which includes automatic traffic loading features to model and analyse the SM1600 traffic model. The SM1600 traffic model was developed to represent the traffic of the future and an economic analysis was a heavy influence in the derivation of the loading model.

The Kirkcaldie and Wood (2008) report further concludes that:

- Aligning the Bridge manual to AS5100 by supplements is more complicated than may at first appear, as significant differences arise in many areas including seismic design, design live loading and slab design. Without extensive supplements the AS5100 does not meet many of the New Zealand design requirements. It would be cumbersome and rather impractical to carry out design using an extensively supplemented document.

- It would be very desirable to have bridge design standards common to New Zealand and Australia. Basic differences of practice (eg. live loading, seismic) could be covered by separate subsections, as has been done in AS/NZS1170).

This report contains a number of subjective views on the potential adoption of the AS5100. It clearly indicates that adopting or harmonising the AS5100 to New Zealand conditions is desirable; however, this approach would be cumbersome, impractical and require considerable resources. The report recommends that although the *Bridge manual* should be retained, a major revision should be carried out to update relevant sections identified in the report and to move towards harmonisation with the AS5100, with specific reference to monitoring and identifying new developments and design procedures of other overseas bridge standards.

A4.4 Vehicle Dimensions and Mass Rule change: Development of bridge design loading for HPMVs

This Opus (2011) report describes the findings of an investigation into the potential increase of vehicle live loading on highway bridges in New Zealand, through the implementation of the VDAM Rule Amendment 2010. The amendment allowed the passage of HPMVs on the New Zealand road network, subject to the issuing of a permit.

The investigation utilised the existing statistical WiM data from the previous 2001 Opus report, described in section A4.2 above, and substituted theoretical HPMV vehicles into the data set.

The scope of the investigation was to confirm the loading arrangement for HPMVs in relation to the existing HN-HO-72 traffic model for assessment of bridge spans extending up to 100m. The study was broken down into the following five stages:

Stage 1: Review the previous study into bridge manual load factors (Opus 2001).

Stage 2: Review and confirm critical HPMV axle weights and configurations through analysis.

Stage 3: Investigate the appropriateness of the current assumption that Class 1 vehicles are represented by 0.85 HN, utilising actual (WiM) data from the Opus (2001) report.

Stage 4: Review the work done by Opus that identified that 0.95 HN is an appropriate design load for bridge spans up to 20m.

Stage 5: Develop loading for 0m–100m spans to review the ability of current design standards to accommodate the increase in live loading through the presence of HPMVs at the:

- ULS - recalculation and review of beta factors to incorporate increased loading through HPMV vehicles
- SLS – analysis of the increased load effects through HPMV loading.

The report makes the following conclusions:

At the ULS:

- Based on the current *Bridge manual* load factors, no changes are required to the ULS load factors for design and assessment loading.
- 0.85 HN is an appropriate representation for Class 1 loading for assessment of existing bridges.
- 0.90 HN is an appropriate representation for HPMV loading for assessment of existing bridges for spans up to 25m.
- 0.95 HN is an appropriate representation for HPMV loading for assessment of existing bridges for spans greater than 25m.

At the SLS:

- The introduction of HPMV vehicles would represent increased loading on the network. If the Transport Agency was willing to accept a potential decrease in the durability and life span of a structure through the introduction of HPMV vehicles, then the SLS design live load factor could be retained at 1.35.
- If the Transport Agency wished to cater for the potential increase in live loading through the introduction of HPMV vehicles, the SLS design live load factor could be increased from 1.35 to 1.5.
- If the Transport Agency wished to undertake further study prior to considering any change to the current SLS design live load factor, this be achieved through a more detailed assessment of actual bridge loading, utilising the latest WiM data.
- The assessment load factor for the SLS should be increased from 0.85 HN to 1.35 HN (inclusive of the live load factor), in order to reflect the actual loading present on the road.

The report describes the analysis carried out and the assumptions considered regarding HPMVs and makes the following observations:

> *...a series of 10 critical axle combinations have been developed. These represent theoretical vehicles (rather than actual vehicles) and were created by putting the maximum axle weights on each of the axles, and minimising the axle spacing to create the worst combinations of axle spacing possible under the new HPMV VDM rules.*

> *The static and moving load data from the original 2002 WIM study included overweight axle loads, which represent actual vehicles using the road. In contrast, the theoretical HPMV axle loads are based upon maximum permitted weights as defined in the VDM Regulations (2010 Amendment).*

Thus, no 'overloading' of the HPMV axles was considered. Based upon historical and more recent WiM data on actual vehicles using the road, the overloading of HPMV vehicle axles is possible. Such an increase in HPMV axle loading would further increase the resultant bending moments and shears.

The axle combinations used to represent the HPMV vehicle in the analysis did not include any potential overloading of the axles except for the assessment of the 'β' factors where the axle loads were simply factored up to carry out a sensitivity analysis. Based on the WiM data collected in 2000, there is a prevalence of overloaded class 1 vehicles on New Zealand roads. In addition, the selection of a theoretical vehicle with maximum axle loads at each axle with minimised axle spacing to represent a HPMV is based on little evidence of actual loading or WiM data. These assumptions are suitable for estimation purposes to assess the increased loading on the network or potential overloading of bridges, although the selection methodology to derive the axle loads and combinations does not confirm the accuracy of the actual loading or overloading of HPMVs on bridges and a more detailed assessment may derive alternative conclusions.

A4.5   NZ Transport Agency overweight permit manual (2005)

The NZ Transport Agency (2005) *Overweight permit manual* is a guideline for the issue of permits for vehicles that exceed the legal mass limits. The guideline covers the administration, procedures and policies for issuing permits. The version of the manual as published on the Transport Agency website is referenced as the first edition published in 2005 and provides reference to the legal mass limits located in the Land Transport Rule: Vehicle Dimensions and Mass 2002. The Rule was amended in 2010 to introduce HPMVs to the road network. To operate a HPMV, which is designed to carry a divisible load, a permit is required where vehicles are restricted to approved routes and must comply with the 2010 Amendment to the Rule. A draft version of the HPMV manual is published on the Transport Agency website for public review[1]. The permit systems for indivisible overweight and over dimension loads are unchanged and a permit is required for any vehicles which exceed the limits established in the Vehicle Dimensions and Mass Rule 2002.

The Transport Agency has a well-documented system for heavy vehicle permit analysis, including bridge assessments. The bridge assessment software was developed inhouse over a number of years and is now part of a web-based permit checking system known as O-Permit. O-Permit draws upon bridge data contained in the bridge asset management database. The bridge assessment software uses the capacity approach. The capacities for each bridge have been previously calculated and are stored in the asset database. Local authorities can decide whether their information is stored with the Transport Agency; however, most local authorities have limited knowledge of their bridge assets.

Load factors and the DLA are built into the system; however, the Transport Agency is currently revising this so that the system can be updated and made more flexible in order to accommodate the equivalent of HPMVs.

O-Permit is not an entirely automated permit checking system, but can be used in various steps of the checking procedure instead of manual calculations. Currently O-Permit, supplemented with manual calculations, is used for checking bridges for HPMVs.

A5   Australia

A5.1   Australian bridge design standard AS5100

A5.1.1   General

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[1] www.nzta.govt.nz/vehicle/your/hpmv/draft-manual.html

The AS5100 was first published as an Australian standard in 2004 and sets out the criteria for the design of new road, railway and pedestrian bridges of conventional form and with spans up to approximately 100m.

Part 2 of the AS5100 specifies the design loads to be adopted including road traffic. Part 7 of the AS5100 specifies the requirements for load rating of existing bridges.

It is noted that the AS5100 is currently under review.

A5.1.2   Traffic load

The traffic loading model of AS5100.2 includes the SM1600 loading which represents the W80 wheel load, A160 axle load, M1600 moving traffic load and the S1600 stationary traffic load. The design code also incorporates the effects of special vehicles through the specification of heavy load platforms HLP320 and HLP400. The loading configurations for these vehicles are shown in figures A.4, A.5 and A.6.

Figure A.4   M1600 moving traffic load
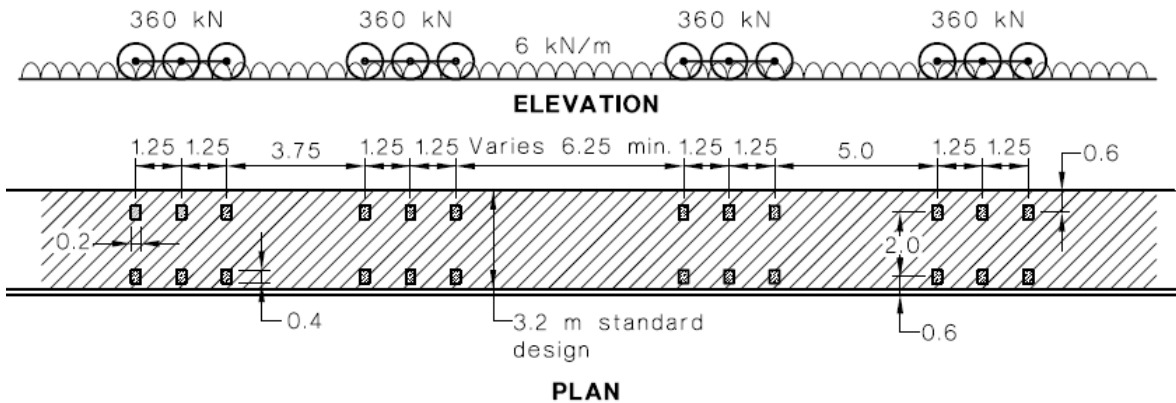
The M1600 moving traffic load consists of a uniformly distributed load of 1.875kPa which equates to a 6kN/m load for a design lane width of 3.2m. The traffic load model also consists of four tri-axle sets, each set consisting of three axles with an axle load of 120kN to give a total load of 360kN. The axle spacing within the tri-axle set is a constant 1.25m. The spacing between the tri-axle sets varies between 3.75m, 5m and 6.25m. The middle group has a variable spacing with a minimum of 6.25m. The variable spacing is more specific to continuous bridges where the spacing is adjusted to gain the most adverse effects. The uniformly distributed load may also be continuous or discontinuous and of any length to produce the most adverse effects. This is also more applicable for continuous bridges.

Figure A.5   S1600 Stationary traffic load
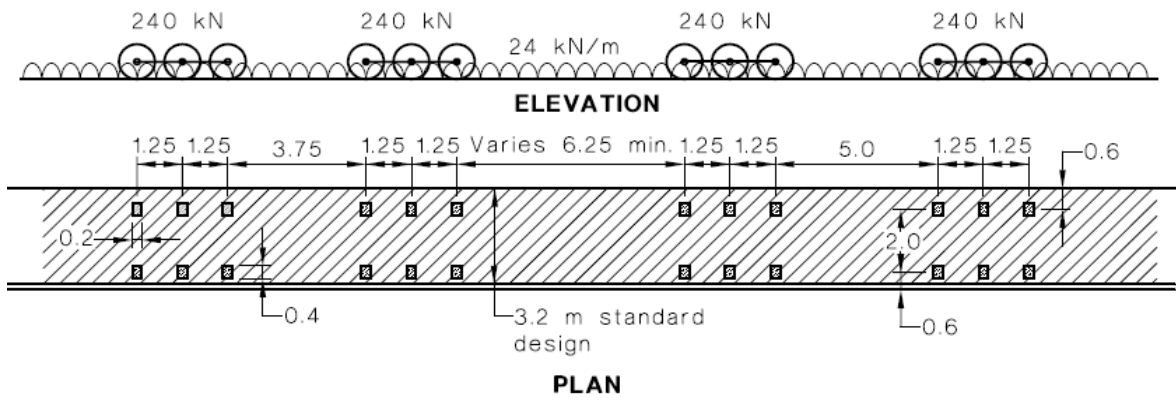
The S1600 stationary traffic load consists of a uniformly distributed load of 7.5kPa which equates to a 24kN/m load for a design lane width of 3.2m. The traffic load model also consists of four tri-axle sets, each set consisting of three axles with an axle load of 80kN to give a total load of 240kN for the tri-axle set. The axle spacing within the tri-axle set is a constant 1.25m. The spacing between the tri-axle sets varies between 3.75m, 5m and 6.25m. The middle group has a variable spacing with a minimum of 6.25m. The variable spacing is more specific to continuous bridges where the spacing is adjusted to gain the most adverse effects. The uniformly distributed load may also be continuous or discontinuous and of any length to produce the most adverse effects. This is also more applicable for continuous bridges.

The A160 axle load represents an individual heavy axle and consists of two wheel loads of 80kN to give a total axle load of 160kN.

The W80 wheel load is an 80kN load distributed over a contact area of 400mm x 250mm (800kPa).

Figure A.6   Heavy load platform traffic load
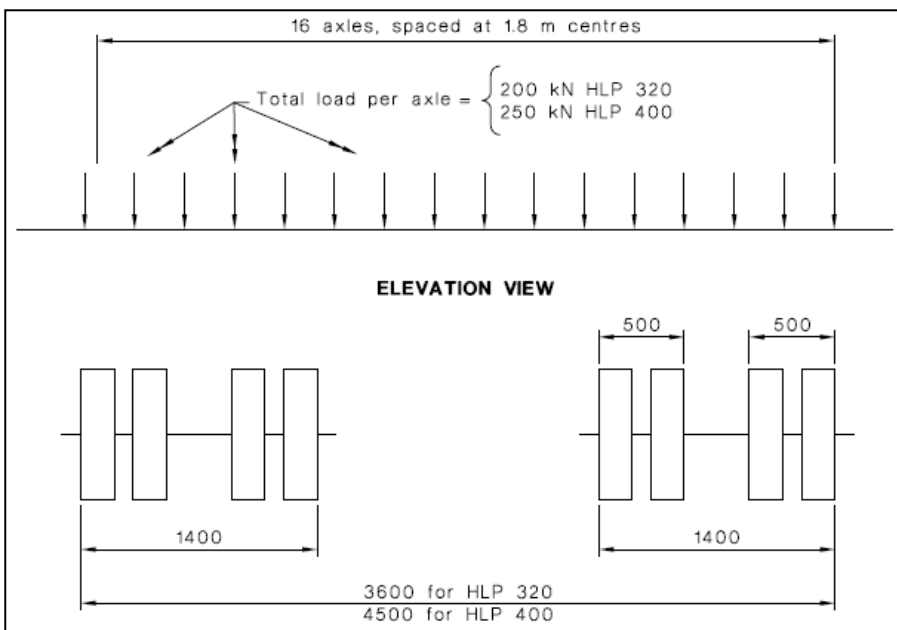
The heavy load platforms HLP320 and HLP400 are special vehicles which are distributed over two design lanes. Each vehicle consists of 16 axles and each axle consists of eight wheel loads. The axle spacing is a constant 1.8m. The loading of the heavy load platform is typically located in the centre of the bridge carriageway and the loading generally falls within the envelope of the SM1600 loading. However, in some cases the HLP loading will govern.

A5.1.3   Lane width and number of lanes

The design lane width for the traffic loading is specified as 3.2m wide.

The number of design lanes is determined by dividing the width of the carriageway between traffic barriers by the standard lane width of 3.2m and rounding down to the nearest integer.

A5.1.4   Lane modification factors

Where more than one lane is loaded, lane modification factors are applied to the accompanying lanes to take into account the reduced probability that extreme loads will occur simultaneously in all lanes. The lane modification factors are shown in table A.13.

Table A.13   Lane modification factors

Number of lanes loaded	Lane modification factor
1 lane	1.0
2 lanes	1.0 for first lane 0.8 for second lane
3 lanes or more	1.0 for first lane 0.8 for second lane 0.4 for third lane and subsequent lanes

The lane modification factors also take into account a reduction in the dynamic load effects as a multi-presence of vehicles on the bridge will not produce vibration effects in the same phase.

A5.1.5   Dynamic load allowance

The dynamic load allowance (DLA) is applied as a proportional increase to the traffic load to model the dynamic effects of moving vehicles. The magnitude of the DLA is dependent on the vehicle type used in the analysis as shown in table A.14. The DLA is only applied to the vertical loads and is not applied to centrifugal or braking forces.

Table A.14   Dynamic load allowance

Loading	Dynamic load allowance
W80 wheel load	0.4
A160 axle load	0.4
M1600 tri-axle set	0.35
M1600 load	0.3
S1600 load	0.0
HLP loading	0.1

A5.1.6   Horizontal loads

Braking forces

The braking forces are applied to the bridge in the longitudinal direction and are calculated from the maximum effects of the following scenarios:

a)   Single vehicle stopping

$F_{BS} = 0.45W_{BS}$ where W_{BS} = load due to a single lane of M1600 traffic load.

$200kN < F_{BS} < 720kN$

b)   Multi-lane moving traffic stream stopping

$F_{BM} = 0.15W_{BM}$ where W_{BM} = load due to multiple lanes of the M1600 traffic load including lane modification factors.

Centrifugal forces

The centrifugal force is calculated as a proportion of the M1600 moving traffic load and is based on the radius of the road and the corresponding design speed of the road. The centrifugal forces are determined from the following equations:

$F_c = (V^2/rg)*W_c$ where V is the design speed (m/s), r is the radius of curve (m) and g is the acceleration due to gravity (9.81m/s²)

$F_c \leq (0.35 + \theta)* W_c$ where θ is the superelevation of the road expressed as a ratio (ie 4% is 0.04)

The centrifugal force is assumed to act at deck level and is applied in accordance with the distribution of load in the M1600 moving traffic load.

A5.1.7 Load factors

The load factor for all design road traffic loads, braking loads and centrifugal loads for the SLS is 1.0.

The load factor for the W80, A160, M1600 and S1600 design road traffic loads, braking loads and centrifugal loads for the ULS is 1.8.

The load factor for the HLP design road traffic loads for the ULS is 1.5.

The following load combinations include the road traffic loading:

Ultimate limit state:

a)   ULS permanent effects + ULS thermal effects + SLS traffic loads

b)   ULS permanent effects + SLS thermal effects + ULS traffic loads

c)   ULS permanent effects + ULS flood load + SLS traffic loads

Serviceability limit state

For the SLS combinations, a number of transient loads may be present at any one time and the following equation represents the combination of loading where the worst combination is determined by the designer.

Load combination = SLS permanent effects + SLS design load for one transient effect + k(SLS design load for one or more other transient or thermal effect)

Where:

    k = 0.7 for one additional effect or 0.5 for two additional effects.

A typical SLS load combination containing the most adverse traffic loads would be:

1   SLS permanent effects + SLS traffic loads + 0.7 (SLS thermal effects)

2   SLS permanent effects + SLS traffic loads + 0.7 (SLS pedestrian loads)

3   SLS permanent effects + SLS traffic loads + 0.5 (SLS thermal effects + SLS pedestrian loads)

4   SLS permanent effects + SLS traffic loads + 0.5(SLS thermal effects + SLS pedestrian loads + SLS wind loads)

A5.1.8 Evaluation

The method of evaluation for Australian bridges contained in AS5100.7 involves procedures for rating the safe load capacity of a bridge for its defined remaining service life. The concept of rating is based on the limit states design principle that the assessed minimum strength capacity of the bridge is greater than the assessed maximum load applied. Both SLS and ULS capacities are considered. Rating relates primarily to the live load condition, including dynamic effects. The procedure is to rate the available live load capacity of the bridge compared with the effects of a nominated rating vehicle. This includes the SM1600 vehicle or some other specific live load configuration. The rating of a bridge is carried out for a specific live load and the effects of the rating loads are determined using the loads and configuration specific to the nominated rating vehicle.

To calculate the rating of a bridge, the factored live load effects of the nominated rating vehicle are compared with the factored strength of the bridge, after subtracting the strength capacities required to

meet the factored dead load and superimposed dead load effects and the parasitic, differential temperature and differential settlement effects. So the rating factor indicates the ability of the bridge to carry the nominated live load and the factor is a proportion of the nominated rating vehicle.

The following equation is used to determine the rating factor of bridges:

$$RF \leq \frac{\emptyset R_u - (\gamma_g S_g + \gamma_g S_{gs} + S_p + S_s + S_t)}{\gamma_L (1 + \alpha) W S_L}$$

where:

$\emptyset R_u$ is the calculated ultimate capacity including capacity reduction factor, $\emptyset$.

$\gamma_g, \gamma_{gs}, \gamma_L$ are the load factors for dead load, superimposed dead load and live load.

$S_g, S_{gs}, S_p, S_S, S_t, S_L$ are the load effects for dead load, superimposed dead load, prestress (or parasitic) effects, differential settlement and live load.

α is the DLA

W is the sum of the lane modification factors.

To describe the equation more simply the rating factor is equal to:

$$RF = \frac{Available\ Capacity\ for\ live\ load\ effects}{Live\ load\ effects\ of\ the\ nominated\ vehicle}$$

A5.1.9   Background

General

The Australian standard AS5100 was preceded by the Association of Australian State Road Authorities (Austroads 1992) *Bridge design code*.

The traffic load models, load factors, DLA and other clauses in the AS5100 were derived through a number of studies and research projects carried out to improve and upgrade the previous versions of Australian bridge design codes. The history and basis of the derivation of the specific clauses related to the traffic loading and evaluation are discussed in this section.

Traffic load

The design traffic loading specified in the *Bridge design code* consisted of a 7t wheel load (W7), a 44t truck (T44), a 1.25t/m uniformly distributed lane load combined with a 15t concentrated load (L44) and a 320t and 400t heavy load platform vehicle (HLP320 and HLP400). The T44 truck loading remained unchanged since 1976. The T44 truck loading model was derived from the AASHTO H20-S16 loading by increasing the load by approximately 35% and replacing the drive and trailer axles with tandem axle sets (Heywood et al 2000).

With the introduction of the AS5100, the loading model was revised to provide for the maximum anticipated increases in vehicle size and weight over the life of the bridge. WiM studies indicated that effects equivalent to the T44 loading on Australian bridges were being experienced on a daily basis and the T44 vehicle was underestimating the average extreme daily events (Heywood 1995b).

Heywood et al (2000) also concludes that a review conducted by the National Road Transport Commission confirmed bridges were limiting the productivity of the road transport system. Developments of transport technology had led to the implementation of B-doubles on the road network and B-triple vehicles were undergoing successful trials at the time of development of the new standard.

The SM1600 loading model was developed to ensure new bridges would not become impediments to future productivity enhancements in road transport and the following considerations were included in the development of the SM1600 loading (SA 2007):

- the freight task as a function of the density of the freight

- the costs and benefits associated with heavier and longer vehicles

- the likely limits associated with vehicle technology and safety issues

- the environmental consequences of larger vehicles

- the cost of upgrading existing bridges to withstand an increased design live loading.

Economic studies (Gordon and Boully 1997) showed that the increased cost of bridges was less than 5%, which was estimated to account for less than 0.25% of road authority expenditure. These studies, which included increased cost of bridges, pavement costs, vehicle performance and safety, showed that the marginal costs associated with providing stronger bridges were small in comparison with the benefits that could be achieved from a more efficient transport system.

Studies of representative vehicles from WiM data (Heywood 1995a) derived two vehicle models, the T55-1221 and T90-12223, to represent the traffic stream on short-span bridges up to 40m. The methodology used for analysing the load effects from the WiM data comprised consideration of six cases of bending moments and shear forces encompassing single spans and twin continuous spans for a range of span lengths. The T55-1221 vehicle model was developed to represent general traffic and comprised two single axles and two groups of tandem axles with a total mass of 55t. The T90-12223 vehicle model was developed to represent road trains and consisted of a single axle, three groups of tandem axles and one tri-axle set with a total mass of 90t.

Studies of heavy vehicles and bridge loadings (Pearson and Bayley 1997) defined a series of vehicles based on the average density of the vehicle plus freight [mass/ (length x width x height)]. Two families of vehicles featured 40t axle sets. These became known as L58 (0.58t/m³) and S73 (0.73t/m³) vehicles. It is expected that future vehicles will be limited by volume restrictions rather than weight and will have an increased number of axles to minimise effects on pavements. The L58 and S73 vehicles are believed to be an upper limit in terms of the available freight task, vehicle technology, safety and pavement damage.

Further studies (Heywood et al 2000) indicated that the average extreme daily effects induced in bridges by a single lane of traffic could be considered in three groups.

1   Slightly overloaded individual wheels, axles and axle sets (A to B in figure A.7).

2   Legally loaded groups of axles or entire vehicles in which the distance between axle sets is at a minimum (B to C in figure A.7).

3   Queues of stationary vehicles (C to E in figure A.7) in which point E is about 60% of point F, where point F corresponds to the load per unit length of a queue of legally loaded vehicles (B-triples). The value of 60% takes into account the space between vehicles and the mix of cars and partially loaded heavy vehicles in the traffic stream.

Figure A.7   Vehicle mass in relation to loaded length (Heywood et al 2000)
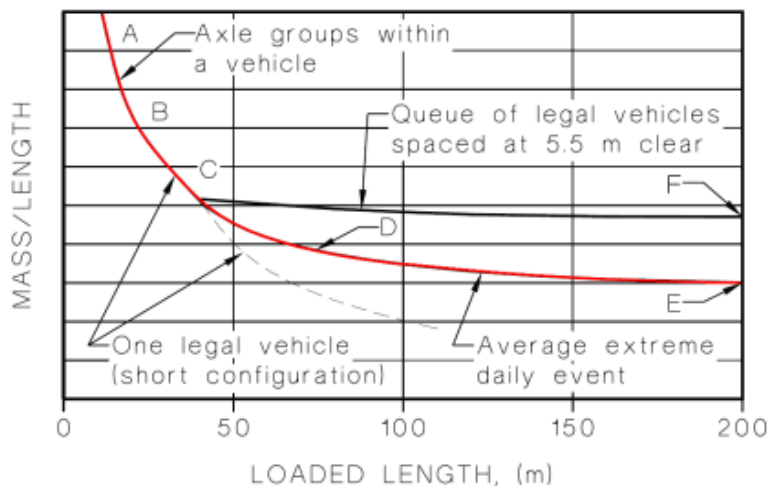
The SM1600 traffic model was developed to make provision for potential future increases in legal load limits. In addition, the model also includes a close representation of the full spectrum of vehicle configurations and traffic patterns. The loading model was designed to induce effects in bridges equivalent to the average extreme daily effects induced by the traffic stream.

W80 WHEEL AND A160 AXLE LOADING

The W80 wheel and A160 axle loads ensure that local bridge elements are sufficiently strong to support an overloaded wheel or axle. Single axles have the highest probability of overloading and combining this with asymmetrical loading results in a higher loading than the nominal. WiM data showed that the average extreme daily axle load is 20% greater than the nominal load. A 40t tri-axle gives an axle load of 133kN and factoring this up by 20% results in the 160kN axle load and 80kN wheel load.

M1600 MOVING VEHICLE LOADING

Evidence extracted from WiM data indicates that heavy vehicles travel at highway speeds with as little as 5m between axles and often less than 9m from another vehicle. The M1600 model simulates the effects induced by the S73 and L58 vehicles in a moving traffic stream which represent an individual heavy combination vehicle or two heavy vehicles together in the same lane with an accompanying stream of traffic. The M1600 vehicle model is equivalent to 40t (392kN) axle groups which were downgraded to 360kN with the addition of the uniformly distributed load. The M1600 vehicle features 4 x 360kN tri-axle sets and a 6kN/m uniformly distributed load that is placed on and under the vehicle. The central axle spacing is variable and permits the model to simulate the effects of two vehicles in adjacent spans. The 6kN/m load represents the moving traffic stream that accompanies a very heavy combination vehicle and also provides protection for continuous bridges. The uniformly distributed load follows the same pattern as the S1600 loading. The M1600 vehicle loading is representative of points B to D in figure A.7.

S1600 STATIONARY VEHICLE LOADING

Short B-triples and triple trailer road trains constituted the most intense loading of the S73 and L58 vehicles and were used to define points C and F in figure A.7 and with point E taken as 60% of point F which was based on Monte Carlo simulations of queued traffic. A Monte Carlo simulation study was carried out based on WiM data and indicated that the relationship between total load (M) and loaded length (L) corresponding to average extreme daily queue of stationary traffic was of the form $M = aL + b$, where a and b are constants. In the case defined by the S73 vehicles, the average extreme daily load

applied to a length of bridge is M = 24L + 1090kN, which corresponds to a 1090kN truck model and a 24kN/m uniformly distributed load. The S1600 stationary traffic loading features 4 x 240kN (960kN) tri-axle sets spaced similarly to the M1600 vehicle and includes a 24kN/m uniformly distributed load. The S1600 vehicle is representative of points C to E in figure A.7.

Lane modification factors

The *Bridge design code* (Austroads 1992) used a lane modification approach where the total load applied to all lanes was averaged and multiplied by the modification factor. This contrasts with the load combination approach where an extreme event is combined with a frequently occurring event. The lane modification factors in AS5100.2 are based on the latter concept. The application of the lane modification factors results in similar total loads as the *Bridge design code*. The factors developed are based on simulations of multi-lane queues of traffic which confirm that the load modification model is appropriate to model the effects of queues in two or more heavily trafficked lanes. The lane modification factors have been further adjusted to account for the fact that dynamic load effects will also be reduced because all vehicles on the bridge will not produce exactly phased vibration effects.

Dynamic load allowance

The DLA adopted for the AS5100 was based on findings of more than 80 field investigations into the dynamic response of bridges in Australia and New Zealand and related theoretical analysis (Standards Australia 2007). This procedure also reflected the changes in the *CAN/CSA-S6-06 Canadian highway bridge design code* where a more simplified process to determine the DLA factors was presented. The results of the investigations (Austroads 2003) found that higher values of dynamic increment occurred for short-span bridges due to axle hop or body bounce; however, these increments occurred at speeds less than 100km/h. Studies also found there was a trend of decreasing dynamic increment with increased mass, but no strong trend towards dynamic increment and bridge frequency except for bridges in the 1Hz to 4Hz range. The highest contributing factor to dynamic increment was related to the road profile and roughness of the road and the effects were more prevalent in short-span bridges. The studies found that the DLA could be determined more accurately and simply by relating the factor to the type of loading rather than the bridge frequency.

The A160 axle loading and a single M1600 tri-axle loading are generally critical for shorter span lengths. The maximum DLA of 0.4 for the W80 wheel load and the A160 axle load reflect the dynamic increments observed in short-span structures and individual components. The DLA of 0.35 for the single tri-axle loading again reflects the dynamic increments observed in short-span bridges, however, allows for some interaction and the spread of load between axles within the group.

For two or more tri-axle set loadings associated with the M1600 loading, the DLA of 0.3 is specified. This is lower than the 0.35 or 0.4 as the loading generally applies for medium or long-span bridges and is representative of typical values obtained from local testing for natural frequencies less than 10Hz and span lengths greater than 10m.

It is noted that the DLA factors specified for the AS5100 are slightly higher than those specified in the CAN/CSA-S6-06 and this also reflects the allowance for rougher roads in Australia.

As stationary traffic presents no dynamic influence on bridges, the DLA is specified as zero. The DLA for HLP loading is specified as 0.1 which reflects the low dynamic increment of a heavy load travelling at low speeds (up to 10km/h).

Horizontal loads

BRAKING

The derivation of the design braking force was dependent on the capacity of the braking system and the friction between the tyres and the pavement (Standards Australia 2007). There is a maximum braking force that is achievable which is based on maximum deceleration rates. Braking tests carried out in Australia concluded that deceleration rates reduced with increasing weight. There are two scenarios to consider for braking: one is a single vehicle stopping and the other is multiple vehicles stopping. The Australian design rules require braking systems to be capable of decelerating heavy vehicles at a minimum rate of 0.45g; however, six-axle articulated vehicles and B-triples have been tested to decelerate at a maximum of 0.75g and 0.6g respectively. In general, low deceleration rates of braking are a regular in-service condition. The standard braking force for a single vehicle stopping is based on the 0.45g deceleration rate. The other possible maximum deceleration rates of 0.6g and 0.75g are covered by the ULS where 0.45g is factored by 1.8 to give a maximum deceleration of 0.81g.

For bridges with a greater surface area between expansion joints, the braking effects may be experienced by multi-traffic lanes. A deceleration rate of 0.15g is applied to the M1600 moving traffic loading which is based on the deceleration rates adopted in road design.

CENTRIFUGAL FORCES

The centrifugal force expression is based on physics with the equation of force = mv^2/r. The maximum centrifugal force is also limited to the upper limit of lateral acceleration for the rollover of legally loaded vehicles with a low centre of gravity, which is approximately 0.5g. This gives rise to the ultimate limit of 1.8 x 0.35g = 0.63g for no super elevation.

Load factors

The loads specified in the Australian code, generally correspond to the SLS (SA 2007) and thus the load factor for the live load for the SLS is specified as 1.0. The load factor for the ULS live load is set as 1.80 which is a 10% reduction on the 2.0 factor from the Austroads (1992) *Bridge design code*. The reduction is consistent with international trends where it is observed that as the legal axle load limits increase, the proportion of vehicles with potential for overloading decreases (SA 2007). This is due to the weight of vehicles being limited by their volume rather than their axle mass. The 1.8 load factor also has the potential for the higher dynamic increments associated with roads with a rough road profile.

A6   United States of America

A6.1   AASHTO LRFD bridge design specification

A6.1.1   General

The first edition of the *AASHTO LRFD bridge design specifications* was published in 1994 and incorporated the most recent load and resistance factor design (LRFD) philosophy. From 1994 to 2007, bridge engineers had a choice of using the latest AASHTO LRFD specification to guide their designs or the long-standing AASHTO load factor design philosophy. In 2007, the Federal Highway Administration and the states adopted the LRFD standards for incorporation in all new bridge designs after 2007. Since the first edition in 1994, the specification has subsequently been amended in 1998, 2004, 2007 and 2010 and 2012.

The provisions in the current AASHTO (2012) specification are intended for the design, evaluation and rehabilitation of both fixed and movable highway bridges. The design provisions of the current specification employ the LRFD methodology with factors developed from the theory of reliability based on current statistical knowledge of loads and structural performance.

Section 3 of the specification outlines the minimum requirements for loads and forces, the limits of their application, load factors and load combinations used for the design of new bridges. The load provisions may also be applied to the structural evaluation of existing bridges.

The evaluation methodology is not included in this specification and a separate manual, the *Manual for bridge evaluation* (AASHTO 2011), includes this procedure. The manual is reviewed separately in section A6.2.

A6.1.2   Traffic loading

The design traffic loading defined in the specification for highway bridges is the HL-93 vehicular live load and comprises a combination of design truck or design tandem with a design lane load.

Design truck

The design truck is shown in figure A.8 and consists of three sets of axle loads. The load on the front axle is specified as 8kip (35.6kN) and the loads on the middle and rear axle are specified as 32kip (142.3kN). The axle spacing between the front and middle axle is fixed at 14ft (4.27m). The spacing of the middle and rear axle may vary from 14ft (4.27m) to 30ft (9.14m) to produce the most adverse effects. The axle width is specified as 6ft (1.83m).

Figure A.8   HL-93 design truck
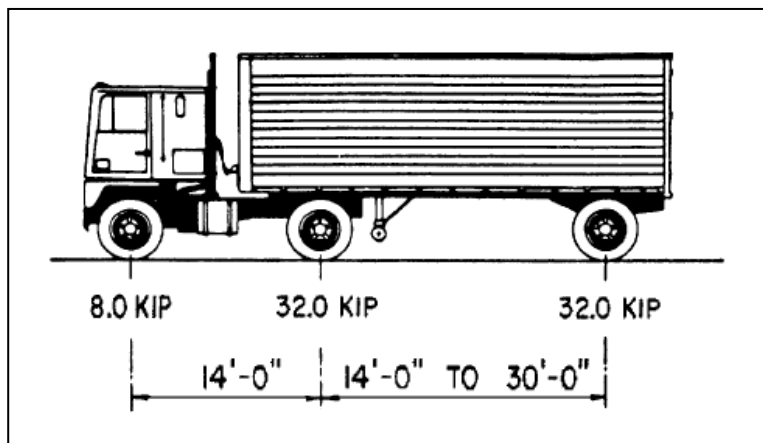
Design tandem

The design tandem is specified as a pair of 25kip (111.2kN) axles spaced at 4ft (1.22m) with an axle width of 6ft (1.83m).

Design lane load

The design lane load consists of a uniformly distributed load of 0.64klf (9.34kN/m)

Wheel loading

The wheel loading is derived from the design truck or tandem axle load which is spread over two or four wheels per axle with each wheel having a contact area of 20in (508mm) wide and 10in (254mm) long. This results in a maximum pressure of 80psi (551kPa).

Live load application

To derive the extreme force effects, the larger of the following live load application are applicable:

1   The design tandem combined with the design lane load (figure A.10)

2   The design truck (with axle spacing set for maximum effects) combined with the design lane load (figure A.9)

3   For negative moment between points of contraflexure and reaction at interior piers, the combination of two design trucks spaced at a minimum of 50ft (15.24m) combined with the design lane load, with all of the forces reduced to 90% (figure A.11).

Figure A.9   HL-93 live loading – truck and lane load
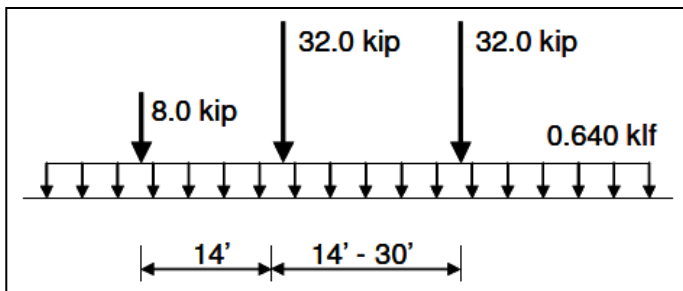
Figure A.10   HL-93 live loading – tandem and lane load
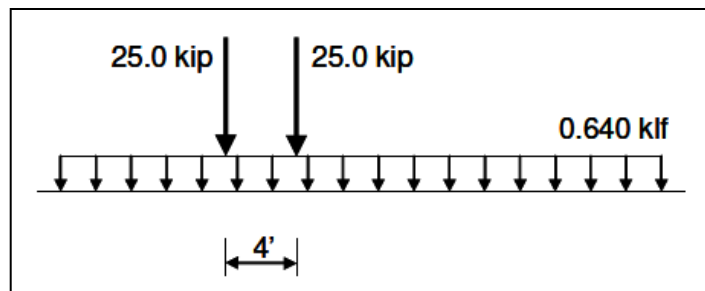
Figure A.11   HL-93 live loading – alternative load for negative moment between points of contraflexure and reaction at interior piers
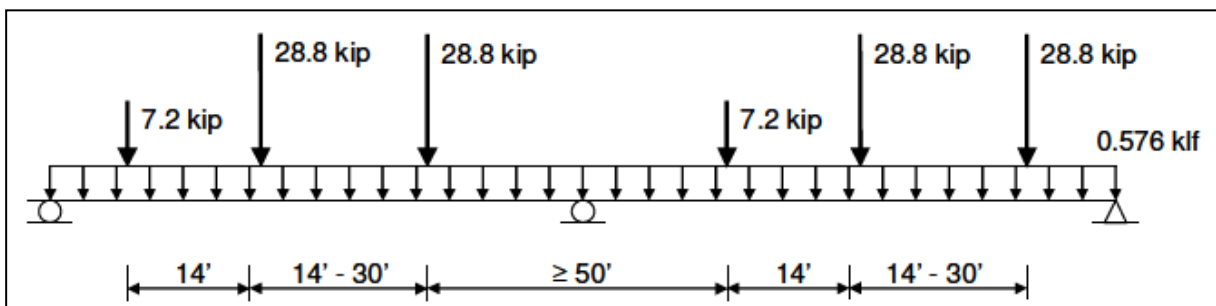
A6.1.3   Lane width and number of lanes

The design lane width for the design traffic loading is defined as 10ft (3.048m).

The number of design lanes is determined by taking the integer part of the ratio:

$$\text{No. of lanes} = \frac{w}{12}$$

where:

w = clear roadway width in feet between kerbs and/or barriers.

If the traffic lanes are less than 12ft (3.66m) then the design lanes equals the traffic lanes. If w is between 20ft (6.10m) and 24ft (7.32m) then the bridge will have two design lanes.

A6.1.4   Multiple presence of live load

The multiple presence factors in table A.15 are applied where either a single lane is loaded or if there are three or more lanes loaded.

The factor is applied equally to all loaded lanes.

Table A.15   Multiple presence factors

Number of loaded lanes	Multiple presence factors, m
1	1.20
2	1.00
3	0.85
>3	0.65

A6.1.5   Dynamic load allowance

The DLA is applied to the design truck or design tandem vehicles but is not applied to the design lane load. The design live load is increased by 33% and is applied to the live load by multiplying the static load by the multiplication factor of 1.33.

A6.1.6   Horizontal loads

Braking forces

The braking force is taken as the greater of:

- 25% of the design truck

- 25% of the design tandem

- 5% of the design truck + lane load

- 5% of the design tandem + lane load.

All design lanes are loaded with the braking force and the multiple presence factors also apply. The load is specified to be applied at 6ft (1.83m) above the road surface and applied in both directions.

Centrifugal forces

The centrifugal force is taken as a proportion of the design truck or design tandem, whichever is greater, and the proportion, C, is defined as:

$$C = f\,\frac{v^2}{gR}$$

where:

v = design speed (ft/s)

f = 4/3

g = gravitational acceleration (ft/s²)

R = radius of curvature of traffic lane (ft).

All design lanes are loaded with the braking force and the multiple presence factors also apply. The load is specified to be applied at 6ft (1.83m) above the road surface and applied in both directions.

A6.1.7   Load factors

The total factored force is specified as:

$$Q = \Sigma \eta_i \gamma_i Q_i$$

where:

η_i = load modifier, which is a factor related to ductility, redundancy and operational classification. This factor is not specific to the actual load and is not discussed further herein.

γ_i = load factor

Q_i = force effects from the loads

The load factor for live load varies depending on the load combination, however for comparison purposes; the following load factors for live load apply which are related to normal vehicular load application:

For the service limit state, the live load factor is specified as 1.0.

For the strength limit state, the live load factor is specified as 1.75.

Table A.16 presents the load factors in the specification for all load combinations. The first column represents the permanent effects, the second column represents the live loads, and the other columns represent other transient loads.

Table A.16   Load combinations and load factors

Load Combination Limit State	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
Fatigue I—LL, IM & CE only	—	1.50	—	—	—	—	—	—	—	—	—	—	—
Fatigue I II— LL, IM & CE only	—	0.75	—	—	—	—	—	—	—	—	—	—	—

A6.1.8   Background

The development of the *AASHTO LRFD bridge design specifications* dates back to 1988 with the commencement of a research program by the National Cooperative Highway Research Program (NCHRP) to develop a limit states design code and to move away from the then current allowable stress design (ASD) approach for the bridge design code. This project, NCHRP Project 12-33, resulted in the publishing of the first edition of the *AASHTO LRFD bridge design specification* in 1994. The work was carried out at the University of Michigan under a subcontract to bridge consulting firm Modjeski and Masters, Inc. A significant part of the project was the development and calibration of the load and resistance factors, as reported in *NCHRP report 368* (Nowak 1999).

Up until this time, AASHTO had used three basic live load models to approximate vehicular live loads: the H or HS design trucks, the design tandem and the design lane loading. The HS design truck is a three-axle truck intended to model a highway semitrailer. The design tandem is a two-axle loading intended to simulate heavy military loadings. The design lane loading primarily consists of a distributed load meant to control the design of longer spans where a string of lighter vehicles, together with one heavier vehicle, might produce critical loadings. In the AASHTO ASD codes, each of these load models was applied individually.

When the original research under NCHRP Project 12-33 was carried out, there was no reliable truck data available in the US to confirm the existing live load model or determine the appropriate load factors to adopt for the new code. Truck survey data for 9250 heavy trucks, collected by the Ontario Ministry of Transportation in the 1970s, was used to determine static bending moments and shear forces for various span lengths (single spans and continuous two equal spans) together with a statistical extrapolation of extreme loads within a 75-year design life. It was assumed that the surveyed truck data represented two weeks of heavy traffic on a two-lane bridge with average daily truck traffic (ADTT) of 1000 in one direction. This work was reported by Nowak and Hong (1991) and indicated that the AASHTO ASD live load models consistently underestimated the load effect of vehicles on the road at the time. Nowak (1993) found that applying the design truck in combination with the design lane loading produced load effect magnitudes comparable to that of measured vehicles.

This work gave rise to the current HL-93 live load model, which is the superposition of a vehicle and a lane load, with the vehicle either the traditional HS20-44 truck or a design tandem, similar to the traditional alternative military loading, whichever produces the greater effect.

Under NCHRP Project 12-33, a live load factor was developed based on around 200 representative bridges from various geographical regions in the US to encompass a wide range of materials, types and span of bridges. Using the reliability analysis results, a live load factor of 1.7 was proposed. In *NCHRP report 20-7/186* (Kulicki et al 2007) the live load factor was increased from 1.7 to the current 1.75 due to the increase in the design ADTT from 1000 to 5000.

Kwon (2011) notes that since NCHRP Project 12-33 has been completed, more reliable truck weight data within the US has been collected through WiM systems and entered in the National Bridge Inventory from which a typical bridge configuration of a specific state can be statistically identified. Thus the load factors in the LRFD specification can be refined for each state based on state-specific truck weights, traffic volumes and bridge configurations. This has been done by the Michigan Department of Transport to increase the live load factor to account for heavy truck traffic in metropolitan areas and has been done in a number of states for rating purposes using WiM data collected in those states.

The current DLA values in the LRFD specification are a result of research undertaken in conjunction with NCHRP Project 12-33 and reported in *NCHRP report 368* (Nowak 1999). It has been demonstrated that in a highway bridge, the actual dynamic response amplitude is a function of a number of factors including bridge

span, continuity, number of girders, slab stiffness, bridge damping, deck roughness, vehicle mass, vehicle speed, number of axles and suspension system. (Hwang and Nowak 1991). The commentary to the LRFD specification gives the basis for the 33% nominated for limit states other than fatigue. Field tests indicated that in the majority of highway bridges, the dynamic component of the response for trucks alone did not exceed 25% of the static response. The specified load combination of design truck and lane load represents a group of exclusion vehicles that are at least 1.33 of those caused by the design truck alone on short and medium span bridges. The 33% is the product of the basic 25% by the 1.33 factor. The commentary also states that the dynamic amplification of trucks has been observed to decrease as the weight of the vehicle goes up, with multiple vehicles and with increased number of axles.

A6.2   AASHTO Manual for bridge evaluation

A6.2.1   General

The *Manual for bridge evaluation* (MBE) was first introduced by AASHTO in 2005. The MBE combines the *Manual for condition evaluation of bridges*, second edition (2000) including interim revisions, with the *Guide manual for condition evaluation and load and resistance factor rating of highway bridges*, first edition including 2005 interim revisions. The MBE first edition superseded both of these manuals. The MBE second edition was published in 2011.

The manual serves as a standard and provides uniformity in the procedures and policies for determining the physical condition, maintenance needs and load capacity of the nation's highway bridges. The provisions in the manual apply to all highway structures which qualify as bridges. The manual assists bridge owners by establishing inspection procedures and evaluation practices that meet the National Bridge Inspection Standards.

Section 6 in the manual discusses the load rating of bridges and is separated into two parts. Part A includes the load and resistance factor method. Part B includes both the load factor method and the allowable stress method. The manual places no preference as to which method should be used.

A6.2.2   Part A – Load and resistance factor rating

General

Part A incorporates provisions specific to the load and resistance factor rating method developed to provide uniform reliability in bridge load ratings, load postings and permit decisions and is consistent with the design philosophy adopted in the *AASHTO LRFD bridge design specifications* (AASHTO 2012).

The methodology for the load and resistance factor rating of bridges comprises three distinct procedures:

1   Design load rating: a first level assessment of bridges based on the HL-93 loading and LRFD design standards, using dimensions and properties of the bridge in its present as-inspected condition. It also serves as a screening process to identify bridges to be load rated for legal loads.

2   Legal load rating: a second level rating which provides a single safe load capacity (for a given truck configuration) applicable to AASHTO and state legal loads. Live load factors are selected based on the truck traffic conditions at the site.

3   Permit load rating: checks the safety and serviceability of bridges in the review of permit applications for the passage of vehicles above the legally established weight limitations.

Loads for evaluation

The vehicular live loading used in the load rating of bridges for the different load rating procedures is shown as the following:

Design load rating: HL-93 design load as per the AASHTO LRFD bridge design specification

Legal load rating: 1) AASHTO legal loads (refer figures A.12 to A.16)

2) The notional rating load or state legal loads (figure A.17)

Permit load rating: Actual permit truck (for spans between 200ft (61m) and 300ft (91.4m) and when checking negative moments in continuous span bridges, an additional lane load of 0.2klf (2.9kN/m) is applied in each lane

Figure A.12   AASHTO legal load – type 3
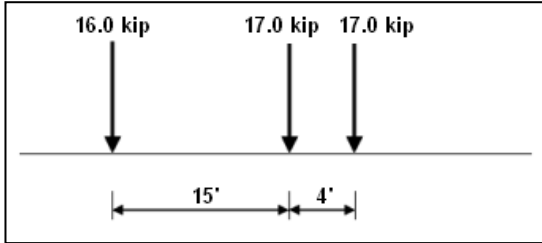
Figure A.13   AASHTO legal load – type 3S2
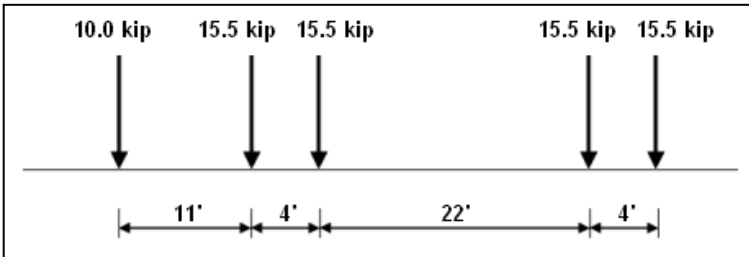
Figure A.14   AASHTO legal load – type 3-3
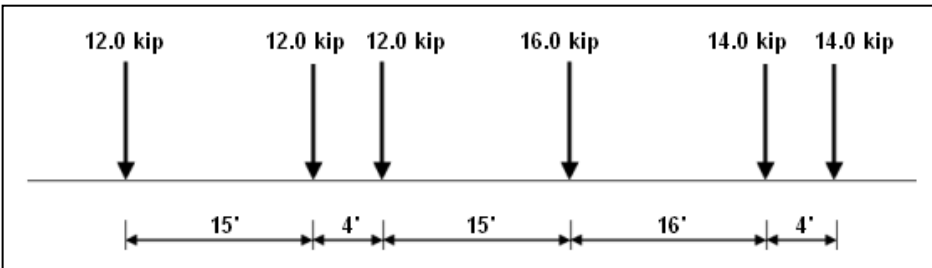
Figure A.15   AASHTO legal load – lane type loading

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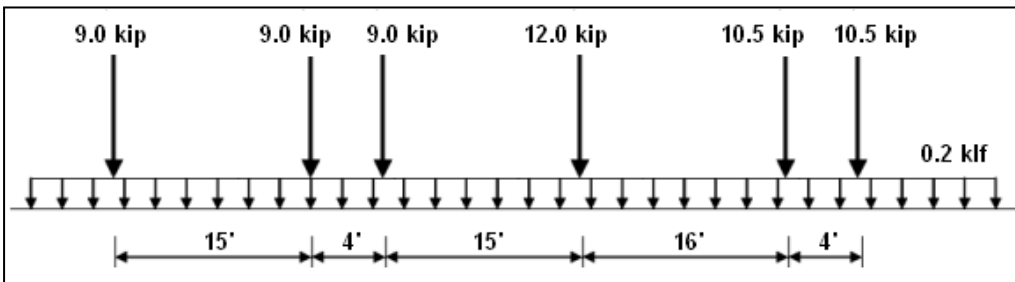
Figure A.16   AASHTO legal load – lane type loading for negative moment and interior reaction
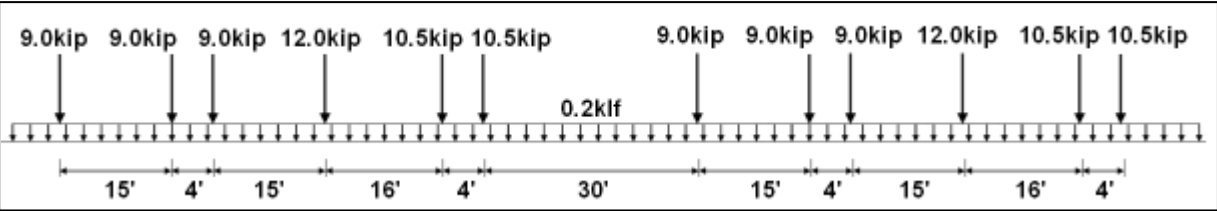
Figure A.17   Notional rating load
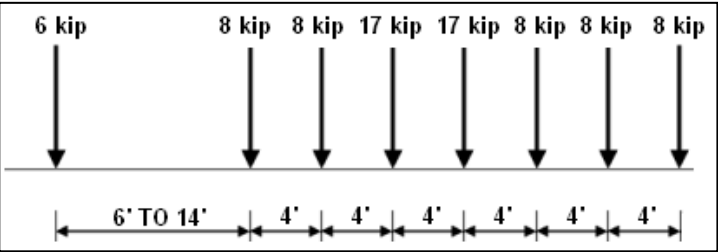
Load factors

The evaluation live load factors for the three rating methods are shown in tables A.17 to A.19 below:

Table A.17   Design load rating

Evaluation level	Load factor
Inventory	1.75
Operating	1.35

The live load factor is dependent on the evaluation level adopted.

The inventory level generally describes a rating to the design level of reliability for new bridges.

The operating level generally describes a rating for a maximum permissible load level to which a structure may be subjected.

Table A.18   Legal load rating

Traffic volume	Load factor	
	AASHTO legal vehicles	Notional rating load
Unknown	1.8	1.6
AADT ≥ 5000	1.8	1.6
AADT = 1000	1.65	1.4
AADT ≤ 100	1.4	1.15

The live load factor is dependent on the traffic volume of the bridge and the rating vehicles used in the evaluation.

Linear interpolation is permitted for other AADT.

Table A.19   Permit load rating

Permit Type	Frequency	Loading Condition	DF^a	ADTT (one direction)	Load Factor by Permit Weight[b]	
					Up to 100 kips	≥150 kips
Routine or Annual	Unlimited Crossings	Mix with traffic (other vehicles may be on the bridge)	Two or more lanes	>5000	1.80	1.30
				=1000	1.60	1.20
				<100	1.40	1.10
					All Weights	
Special or Limited Crossing	Single-Trip	Escorted with no other vehicles on the bridge	One lane	N/A	1.15	
	Single-Trip	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.50	
				=1000	1.40	
				<100	1.35	
	Multiple-Trips (less than 100 crossings	Mix with traffic (other vehicles may be on the bridge)	One lane	>5000	1.85	
				=1000	1.75	
				<100	1.55	

The live load factor is dependent on the permit type, the frequency of the crossings of the bridges subject to the permit, the conditions of mixing with other vehicles, the weight of the vehicle and the traffic volume.

Dynamic load allowance

1   Design load rating

    The DLA is as per the *AASHTO LRFD bridge design specification* and is typically an allowance of 33% or a multiplication of live load by 1.33.

2   Legal load rating

    The DLA for the legal load rating is 33% or a multiplication of live load by 1.33. The DLA is only applied to the vehicle component and not to the lane loading component.

3   Permit load rating

    The DLA for the permit load rating is 33% or a multiplication of live load by 1.33 except for slow moving vehicles (≤ 10 mph) the DLA may be excluded. The DLA is only applied to the vehicle component and not to the lane loading component.

Rating of bridges

To determine the load rating for each component of the bridge the following general expression is used:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)}$$

where:

C is the capacity – where C = Nominal resistance including resistance factors for ULS

C = Allowable stress for SLS

$\gamma_{DC}, \gamma_{DW}, \gamma_P, \gamma_{LL}$ are the load factors

DC, DW are the dead load for structural components and wearing surface

P is the permanent loads other than dead loads

LL is the live load effect

IM is the DLA

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Posting of bridges

If state legal loads exceed the calculated load capacity of the bridge, the bridge must be posted.

The live load to be used for the posting considerations should be any of the AASHTO legal loads or state legal loads.

The following equation is used to establish the safe posting load:

$$Safe\ Posting\ Load = \frac{W}{0.7}\,[(RF) - 0.3]$$

RF = Legal load rating factor

W = Weight of rating vehicle

Where the RF is less than 0.3 the vehicle type is not allowed on the span.

A6.2.3   Part B – Allowable stress rating and load factor rating

Introduction

Part B provides safety criteria and procedures for the allowable stress and load factor methods of evaluation. For the bridge evaluation in accordance with part B, there is a choice of two rating methods and load ratings at the inventory and operating levels can be calculated using either the:

1   Allowable stress method, or

2   Load factor method.

Rating levels

The inventory rating level generally corresponds to the customary design level stresses but reflects the existing bridge and material condition with regard to deterioration and loss of section.

The operating rating load generally describes the maximum permissible live load to which the structure may be subjected.

Rating methods

1   Allowable stress method

   The allowable or working stress method constitutes a traditional specification to provide structural safety. The actual loadings are combined to produce the maximum stress in a member, which is not to exceed the allowable or working stress. The latter is found by taking the limiting stress of the material and applying an appropriate factor of safety.

2   Load factor method

   The load factor method is based on analysing a structure subject to factored loads. Different factors are applied to each type of load, which reflect the uncertainty inherent in the load calculations. The rating is determined such that the effect of the factored loads does not exceed the strength of the member.

Rating equation

The following equation is used to determine the load rating of the structure:

$$RF = \frac{C - A_1 D}{A_2 L (1 + I)}$$

where:

   RF = the rating factor

C = the capacity of the member

D = the dead load effects

L = the live load effects

A_1 = the dead load factor

A_2 = the live load factor

I = the impact factor (also known as the dynamic effects)

For the allowable stress method A_1, A_2 = 1.0 and the capacity, C, is selected depending on specified limits based on the rating level.

For the load factor method A_1 = 1.3 for both levels. A_2 = 2.17 for the inventory level and A_2 = 1.3 for the operating level. The capacity, C, is calculated for each component from the *AASHTO Standard specifications for highway bridges* (AASHTO 2002).

Rating live load

The live load to be used in the load rating is the HS-20 truck or lane loading as specified in the previous *AASHTO Standard specifications for highway bridges* (AASHTO 2002) and is shown in figures A.18 and A.19.

Figure A.18   Standard HS truck

HS20-44	8,000 LBS	32,000 LBS	32,000 LBS
HS15-44	6,000 LBS	24,000 LBS	24,000 LBS

14'   14' TO 30'
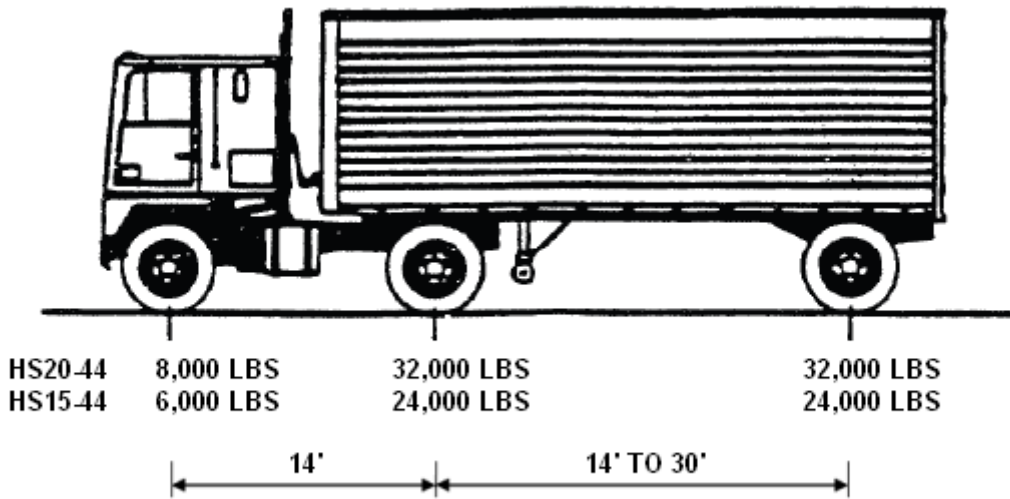
Figure A.19   Standard lane load

CONCENTRATED LOAD — 18,000 LBS FOR MOMENT
26,000 LBS FOR SHEAR

UNIFORM LOAD 640 LBS PER LINEAR FOOT OF LOAD LANE
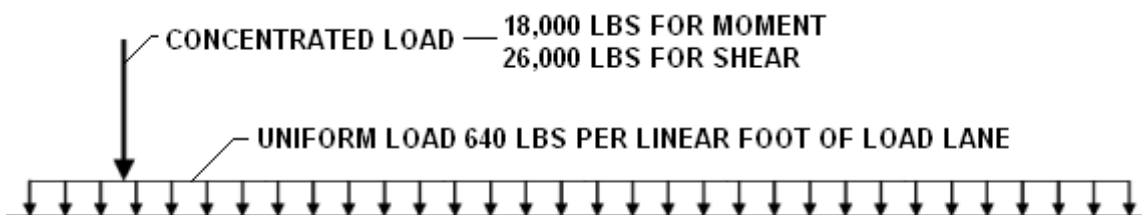
A6.2.4   Background

The AASHTO (2011) *Manual for bridge evaluation* was developed as part of the NCHRP Project 12-46 initiated in 1997. The objective of the project was to develop a manual for the evaluation of the condition

of highway bridges consistent with the design and construction provisions of the *AASHTO LRFD bridge design specifications*, but with calibrated load factors appropriate for bridge evaluation and rating. The associated *NCHRP report* (Moses 2001) presents the derivations of the live load factors and associated checking criteria incorporated in the manual.

Moses (2001) notes that several considerations were involved in recommending the adoption of the AASHTO legal vehicles as the basis for the calculation of legal load bridge ratings rather than the HL-93 model. First, these legal vehicles were familiar to rating agencies and easily converted to tons of legal loading for reporting, and had been used for many years to determine if a bridge required posting for legal loads and to further select posting. Further, the model of the legal vehicles was also easier to express in a posting format.

The recommended evaluation format in the evaluation manual uses the legal vehicles as the nominal live loading configuration of the trucks, needed for computing the bending and shear effects. These load effects are then multiplied by the live load factors, which have been derived from calibration using the reliability indexes reported in Moses (2001). A consistent approach was adopted in calibrating the live load factors. The aim of the calibration was to achieve uniform target reliability indexes over the range of applications, including design load rating, legal load rating, posting and permit vehicle analysis (Moses 2001). Hence the live load factors are different for the three scenarios reflecting the different probabilities of overloading for the different scenarios.

As much as possible, the database of the *AASHTO LRFD bridge design specifications* was utilised. The loading database was based on an extreme truck weight spectra (Nowak 1999).

A7 Canada

A7.1 CAN/CSA-S6-06 Canadian highway bridge design code

A7.1.1 General

The 10th edition (2010) of the CAN/CSA-S6-06 was reviewed.

This code applies to the design, evaluation and structural rehabilitation design of fixed and movable highway bridges in Canada. There is no limit on span length, but this code does not necessarily cover all aspects of design for every type of long-span bridge. This code uses the limit states design approach and reflects current design conditions across Canada as well as research activity since the publication of the previous edition. This code has been written to be applicable in all provinces and territories.

Section 3 of the code specifies loading requirements for the design of new bridges, including requirements for permanent loads, live loads and miscellaneous transitory and exceptional loads. This section specifies loads, load factors and load combinations to be used in calculating load effects for design. The 625kN truck load model and corresponding lane load model are specified as the minima for interprovincial transportation and are based on current Canadian legal loads.

Section 14 specifies methods of evaluating an existing bridge to determine whether it will carry a particular load or set of loads. This section includes new provisions concerning the three level evaluation system, evaluation of deck slabs and detailed evaluation from bridge testing.

A7.1.2 Traffic loading

The design traffic loading for highway bridges is specified as the CL-W loading which consists of the CL-W Truck or the CL-W lane load.

CL-W truck

The CL-W truck is shown in figure A.20 and consists of five idealised axle loads. W represents the gross load of the truck and the axle loading is shown as a proportion of the gross load. The CL-625 truck loading is specified as the minimum for the design of the national highway traffic loading that is generally used for interprovincial transportation. The CL-625 truck consists of a 50kN front axle load, two sets of 125kN axle loads, a 175kN axle load and a 150kN rear axle load. The spacing of the axles from the front axle is 3.6m, 1.2m, 6.6m and 6.6m. The width of the axle is specified as 1.8m.

Figure A.20   CL-W truck loading
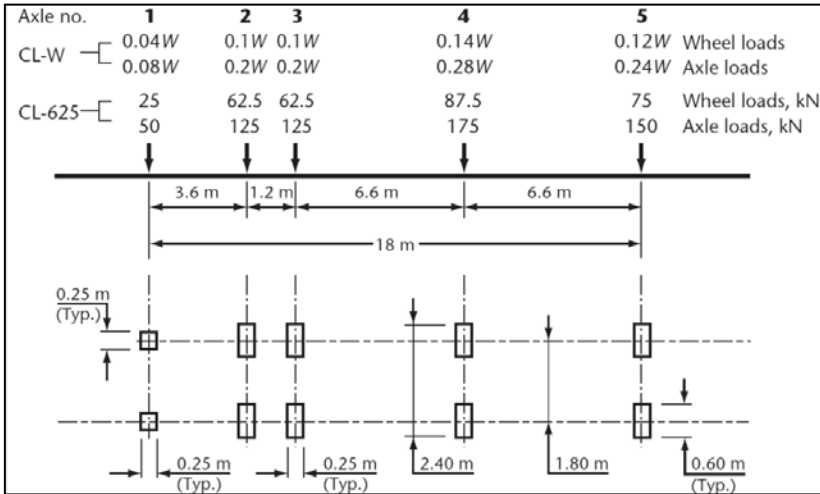
CL-W lane loading

The CL-W lane load consists of the CL-W truck with each axle reduced to 80% of the truck loading (ie a CL-500 truck for a CL-625 lane loading) superimposed with a uniformly distributed load of 9kN/m. The CL-625 lane loading is also specified as the minimum for the design of the national highway traffic loading. The CL-W lane load is shown in figure A.21.

Figure A.21   CL-W lane loading
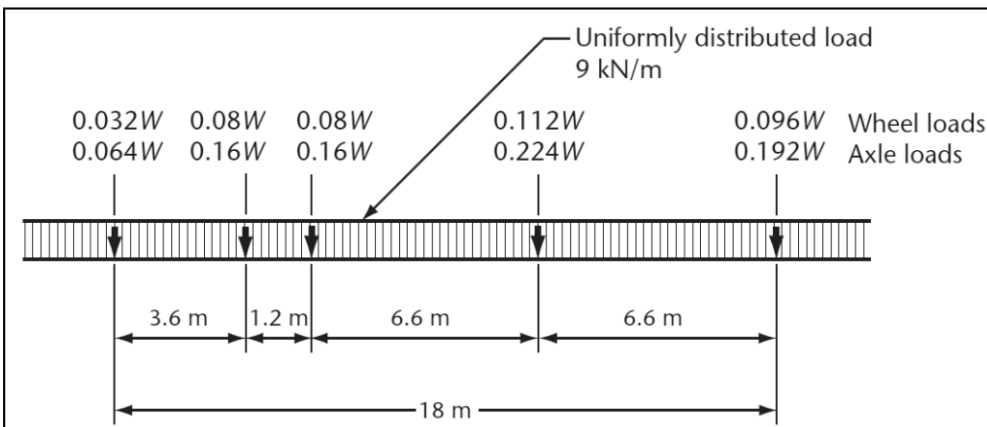
Wheel and axle loading

For the design of local components, the axle loads specified in the CL-W truck loading are used and depending on the component, either axle 2, axle 2 and 3, or axle 4 is used for the loading. The wheel loading is half the axle loading and is applied over an area of 250mm x 250mm for the front axle and

600mm x 250mm for the other axles which gives a max pressure of 400kPa and 583kPa for the front wheel and other wheels respectively.

Live load application

The live load is applied as either the CL-W truck loading or the CL-W lane loading, whichever gives the most adverse load effects. Any axles of the truck loading which reduce the load effects are excluded. Any length of the uniformly distributed load component of the lane loading that reduces the load effects are also excluded.

A7.1.3   Lane width and number of lanes

The clearance envelope for the design truck loading or design lane loading is defined as 3m. The width of the design lane, W_e, is defined as the width of the deck, W_c, divided by the number of lanes, n.

The number of design lanes is determined from table A.21.

Table A.21   Number of lanes

Deck width, W_c	Number of lanes, n
6.0m or less	1
Over 6.0m to 10.0m	2
Over 10.0m to 13.5m	2 or 3*
Over 13.5m to 17.0m	4
Over 17.0m to 20.5m	5
Over 20.5m to 24.0m	6
Over 24.0m to 27.5m	7
Over 27.5m	8

*both should be checked

A7.1.4   Multi-lane loading

Where more than one lane is loading, the following modification factors in table A.22 are applied to the total traffic load depending on the number of lanes loaded:

Table A.22   Multi-lane modification factors

Number of loaded design lanes	Modification factor
1	1.0
2	0.9
3	0.8
4	0.7
5	0.6
6 or more	0.55

A7.1.5   Dynamic load allowance

The DLA is applied to the design truck load but is not applied to the design lane loading. The DLA is applied to increase the truck loads by the proportion of the load specified and can be represented by the multiplication factor of (1 + DLA).

The following DLAs are used:

1   0.50 for deck joints

2   0.40 where only one axle of the CL-W truck is used

3   0.30 where any two axles of the CL-W truck, or axles 1 to 3, are used

4   0.25 where three axles of the CL-W truck or more, except for axles 1 to 3, are used.

A7.1.6   Horizontal loads

Braking forces

The braking force is only considered at the ULS.

The braking force is defined as a 180kN horizontal loading plus 10% of the uniformly distributed load of the lane load from one design lane only and not greater than 700kN.

The braking force is applied at the deck surface.

Centrifugal forces

The centrifugal force is taken as a proportion of the design truck loading. The centrifugal force is applied at right angles to the direction of travel and at 2m above the deck surface.

The proportion is determined by the following equation:

$$Proportion = \frac{v^2}{127r}$$

where:

  v is the design speed (km/h)

  r is the radius of the road (m)

A7.1.7   Load factors

The load factor for live load varies depending on the load combination; however, for comparison purposes the following load factors for live load apply. These are related to normal vehicular load application.

•   For the SLS, the live load factor is 0.90.

•   For the ULS, the live load factor is 1.70.

Table A.23 presents the load factors in the design code for all load combinations, where L represents the live load.

Table A.23   Load combinations and load factors

Loads	Permanent loads			Transitory loads					Exceptional loads			
	D	E	P	L^*	K	W	V	S	EQ	F	A	H
Fatigue limit state												
FLS Combination 1	1.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	1.70	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	1.60	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	1.40	1.00	0.50§	0.50	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.65§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.90§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

A7.1.8   Evaluation

The evaluation of a bridge is carried out to determine whether a bridge will carry a particular load or set of loads. A bridge is evaluated in accordance with one or more of the following methods:

1   Ultimate limit state

    a)   ULS methods using load and resistance adjustment factors

    b)   mean load method

    c)   load testing

2   Serviceability limit state

The evaluation procedure seeks to determine a live load capacity factor, F, which gives the indication of the ability of the bridge to withstand a nominated live load and to determine posting restrictions if required.

Target reliability index factors, β, are included into the evaluation procedure and forms the basis for the determination of load factors to be used in the evaluation process.

Live loading

Bridges are evaluated against a live load which includes the following types of live loading conditions:

1   Normal traffic (three levels)

    a)   evaluation level 1 (vehicle train)

    b)   evaluation level 2 (two-unit vehicles)

    c)   evaluation level 3 (single-unit vehicles)

2   Permit vehicles (four classifications)

    a)   permit – annual or project (PA)

    b)   permit – bulk haul (PB)

    c)   permit – controlled (PC)

    d)   permit – single trip (PS).

Normal traffic – evaluation level 1

A bridge evaluated to level 1 includes bridges that are required to carry vehicle trains in normal traffic. The live loading used for evaluation level 1 is the CL1-W truck or lane load as shown in figures A.22 and A.23. The value of W is 625kN unless approved otherwise by the regulatory authority.

Figure A.22   CL1-W truck loading
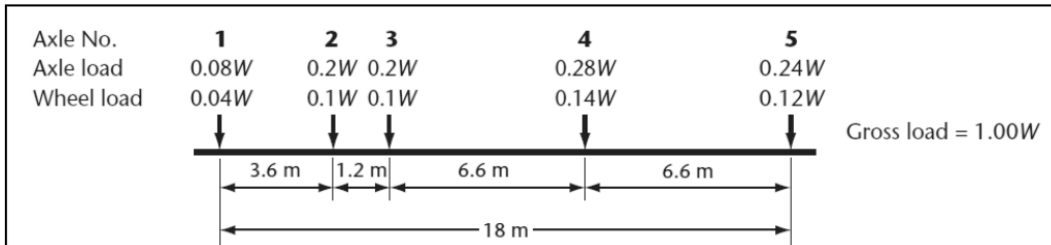
Figure A.23   CL1-W lane loading
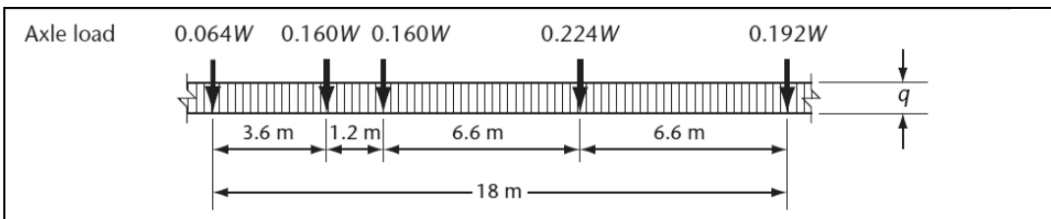
Normal traffic – evaluation level 2

A bridge evaluated to level 2 includes bridges where load restrictions are to apply and the bridge is required to carry two-unit vehicles. The live loading used for evaluation level 2 is the CL2-W truck or lane load. The CL2-W loading includes only the first four axles of the CL1-W loading.

Normal traffic – evaluation level 3

A bridge evaluated to level 3 includes bridges where load restrictions are to apply and the bridge is required to carry single-unit vehicles. The live loading used for evaluation level 3 is the CL3-W truck or lane load. The CL3-W loading includes only the first three axles of the CL1-W loading.

Permit vehicle – PA, PB, PC and PS

A bridge evaluated for a permit vehicle depends on the permit type and the bridge is evaluated against the actual permit vehicle. For PA, PB and PS type permits, the bridge is also evaluated for 85% of the permit vehicle (80% for PB) plus a superimposed uniformly distributed load of 9,8,7 and 7kN/m for highway class A,B,C and D respectively, without DLA.

Live load capacity factor

ULTIMATE LIMIT STATE

For the ULS, the live load capacity factor is calculated by the following equation:

$$F = \frac{UR_r - \Sigma\alpha_D D - \Sigma\alpha_A A}{\alpha_L L(1 + I)}$$

where:

U is the resistance adjustment factor

R_r is the resistance

$\alpha_D, \alpha_{A,} \alpha_L$ are load factors for dead loads, additional loads and live loads

D, A, L are the dead, additional and live load effects

I is the DLA

The live load capacity factor may also be calculated using the mean load method or by load testing.

SERVICEABILITY LIMIT STATE

For the SLS, the live load capacity factor is calculated by the following equation:

$$F = \frac{\sigma_{SLS} - \sigma_D - \sigma_A}{\alpha_L \sigma_L (1 + I)}$$

where:

σ_{SLS} is the maximum stress level for the limit state

$\sigma_D, \sigma_A, \sigma_L$ are the stress values for dead loads, additional loads and live loads

α_L is the SLS load factors for live load effects

I is the DLA.

Bridge posting

The live load capacity factors are used for posting of bridges and the posting depends on the evaluation level and are applied as follows:

1   When F ≥ 1.0 for evaluation level 1, no posting is required.

2   When 1.0 > F ≥ 0.3 for evaluation level 1, triple posting is required with the posting loads for each evaluation level taken from figure A.24 for the appropriate value of F for each level.

3   When F < 0.3 for evaluation level 1 and F ≥ 0.3 for evaluation level 3, single posting to evaluation level 3 is required with the posting load taken from figure A.24 for evaluation level 3.

4   When F < 0.3 for evaluation level 3, consideration is given to closing the bridge.

Figure A.24   Posting loads for gross vehicle weight
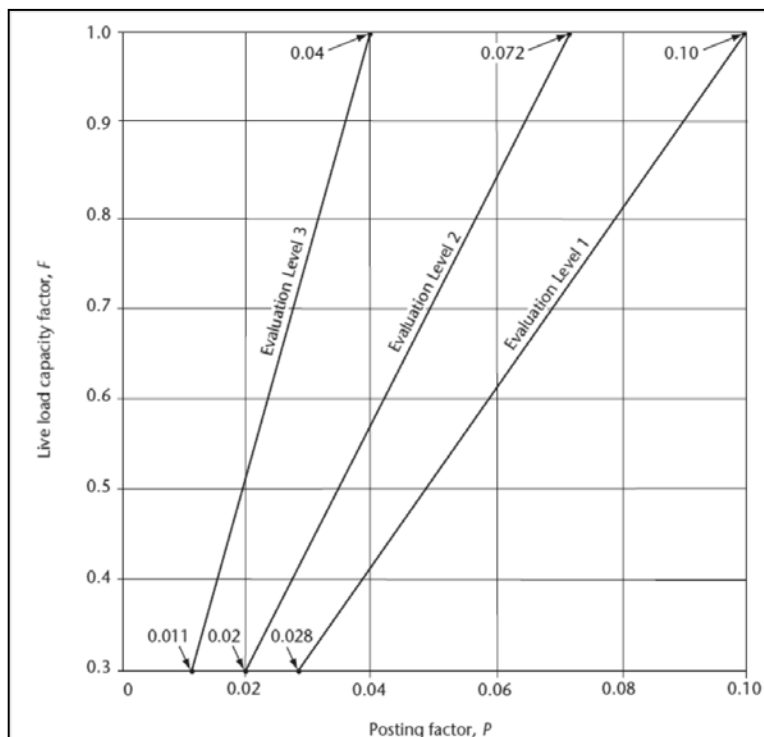
The posting weight limit is in tonnes and is calculated by P x W, where P is taken from figure A.24 and W is taken from the live load used in the evaluation in kilonewtons (ie 625kN for the C1-625 loading).

A7.1.9 Background

The 2006 edition of *CAN/CSA-S6, Canadian highway bridge design code* (with subsequent amendments) supersedes the previous edition published in 2000, which amalgamated and superseded *CAN/CSA-S6-88, Design of highway bridges*, and the Ontario Ministry of Transportation's *OHBDC-91-01, Ontario highway bridge design code*, third edition.

As reported by Agarwal and Cheung (1987), the early Canadian Standards Association (CSA) codes were similar in their specification of a live load model to the AASHTO HS loading model. In 1979, Ontario developed a new highway bridge design code which contained a new live load model based on commercial vehicle weight surveys conducted in Ontario from 1967 to 1975. CSA subsequently developed a live load model designated CS-W comprising a truck loading combined with a lane loading based on simulations performed using actual traffic statistics collected in manual surveys across all Canadian provinces. In determining the load effects from this survey data, bending moments and shear forces were determined for single spans and for two equal and three equal continuous spans for a range of span lengths. This model reflected the regulatory level of vehicle loads in Canada and a uniform live load factor at the ULS of 1.60 was nominated for all spans. The *Ontario highway bridge design code* truck model, however, included maximum observed overloads in Ontario, thus reflecting a load level higher than the regulatory level. The corresponding live load factor at the ULS was 1.4. The amalgamated version of the CAN/CSA-S6 published in 2000 included a ULS load factor of 1.7. This load factor is calibrated to achieve a reliability index, β, of 3.5 which gives a small probability (in order of 1%) of the ULS live load being exceeded in 75 years.

As the commentary to CAN/CSA-S6-06 (2006) explains, the degree and nature of overloads may vary from one province to another and the observations in Ontario may not be directly applicable to other provinces. The live load model for the current CAN/CSA-S6-06 (2006) thus adopted a regulatory load level, but a heavy dual axle was also included in the truck model with some degree of overload in order to make the model more efficient in achieving a uniform reliability for all ranges of span length and all types of bridges. This is a similar approach to that adopted in the AASHTO LRFD specification which consists of the HS truck and, alternatively, a heavy design tandem (Nowak 1993). The minimum standard for the CL-W truck is the CL-625 truck with a gross weight of 625kN.

The CL-W lane load is based on the traffic loading for long-span bridges recommended by the American Society of Civil Engineers Committee on Loads and Forces on Bridges (Buckland 1981).

The background to the DLA nominated in CAN/CSA-S6-06 (2006) is explained in detail in the commentary and is based on results from a number of tests carried out by research between 1964 and 1982.

As explained in the commentary, the philosophy behind section 14 'Evaluation' is to determine a suitable safety level for each element of the bridge under evaluation, which varies with the type of element failure to be expected: more safety is required for an element that fails abruptly; less safety is required for an element that will retain its capacity after failure and may shed its load to other members without collapse. The main parameters in setting the required safety level, defined by the reliability index, β, are the behaviour of the element being considered, the behaviour of the structural system of which the element is a part, and the degree of inspection of the bridge. Inspection is important to ensure that the bridge is indeed in the condition the evaluator is assuming, and to verify that it has carried previous loads without distress.

For the purpose of evaluation, normal traffic is divided into three vehicle categories (Agarwal and Csagoly 1978). A bridge is evaluated separately for each of these three categories of vehicles represented by an evaluation level as follows:

1   Evaluation level CL1-W: vehicle trains consisting of a tractor and more than one trailer, such as a 'B-train'

2   Evaluation level CL2-W: truck/trailer or tractor/semitrailer combinations

3   Evaluation level CL3-W: single unit vehicles.

As noted in the commentary, the design loading model CL-W was developed so that its sub-configurations conveniently represent the three vehicle categories. These have been designated as the CL1-W, CL2-W and CL3-W (Agarwal et al 1993). CL1-W is basically the same as the design loading model CL-W. Note that 'W' is a measure of general traffic on the highway system, not necessarily of traffic permitted on the bridge being evaluated. Hence, a CL1-625 truck weighs 625kN, but a CL2-625 weighs only 475kN because it has fewer axles.

The loading models provide flexibility in selecting the magnitude of the load, W, based on local regulations and conditions of the traffic. Interpretation of the evaluation results, particularly determination of the posting loads described in clause 14.17.2, takes into account the selected value of W. The value of 625 for W is applicable to the national highway network that will generally be used for interprovincial transportation.

The uniformly distributed load in the lane loads represents the mix of traffic in a lane in addition to one heavy truck belonging to a particular category. This mix of traffic does not depend on the category of the 'one truck' and hence, the magnitude of the uniformly distributed load remains unchanged with the evaluation level. However, the characteristic and the load of the traffic mix depend on the traffic volume and composition of the traffic. Therefore, the magnitude of the uniformly distributed load changes with the highway class, increasing with the traffic volume.

A8   Europe

A8.1   Eurocode EN 1990 and EN1991-2

A8.1.1   General

The Eurocode is a European standard which is given the status of a national standard by the member countries by the publication of an identical text or by endorsement. The national standards implementing Eurocodes comprise the full text of the Eurocode (including annexes) and may be followed by a national annex. The national annex may only contain information on parameters left open in the Eurocode for national choice. These are known as nationally determined parameters and are used for the design of buildings and civil engineering works constructed in the country concerned. The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature.

EN 1990 Eurocode (CEN 2005) establishes principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification and gives guidelines for related aspects of structural reliability. It is based on the limit state concept used in conjunction with a partial load factor method. EN 1990 is intended to be used in conjunction with EN 1991 to EN 1999 for the design of new structures and the structural appraisal of existing structures. EN 1990 was first published in 2002 and supersedes ENV 1991-1:1996.

EN 1991-2 Eurocode 1 (CEN 2003) is applicable for the design of new bridges with spans up to 200m. It defines imposed loads (models and representative values) associated with road traffic, pedestrian actions and rail traffic which include, when relevant, dynamic effects and centrifugal, braking and acceleration actions and actions for

accidental design situations such as collisions. EN1991-2 superseded ENV1991-3:1995, in which the outcomes of the original development work for the traffic load models were first published.

A8.1.2   Traffic loading

The design traffic loading consists of four load models which are intended to cover all foreseeable traffic situations to be taken into account for design.

Load model 1

Load model 1 consists of two axle loads and a uniformly distributed load which represent the load effects of the traffic of trucks and cars (figure A.24). The axle spacing is defined as 1.2m and the axle width is defined as 2.0m. The traffic loading model includes dynamic amplification and load modifications for multiple lane loading within the model.

The axle load is defined as: $\alpha_Q Q_k$

The uniformly distributed load is defined as: $\alpha_Q q_k$

where:

α_Q is the adjustment factor defined by the national annex or taken as 1.0.

Q_k is the characteristic axle load which depends on the lane loaded and taken from table A.24.

q_k is the characteristic uniformly distributed load which depends on the lane loaded and taken from table A.24.

Table A.24   Load model 1: characteristic values

Location	Tandem system TS	Uniformly distributed load system
	Axle loads Q_k (kN)	q_k (kN/m²)
Lane number 1	300	9
Lane number 2	200	2.5
Lane number 3	100	2.5
Other lanes	0	2.5
Remaining area	0	2.5

The details of load model 1 are illustrated in figure A.25.

Figure A.25   Application of load model 1

(1) Lane Nr. 1 : $Q_{1k} = 300$ kN ; $q_{1k} = 9$ kN/m^2

(2) Lane Nr. 2 : $Q_{2k} = 200$ kN ; $q_{2k} = 2,5$ kN/m^2

(3) Lane Nr. 3 : $Q_{3k} = 100$ kN ; $q_{3k} = 2,5$ kN/m^2
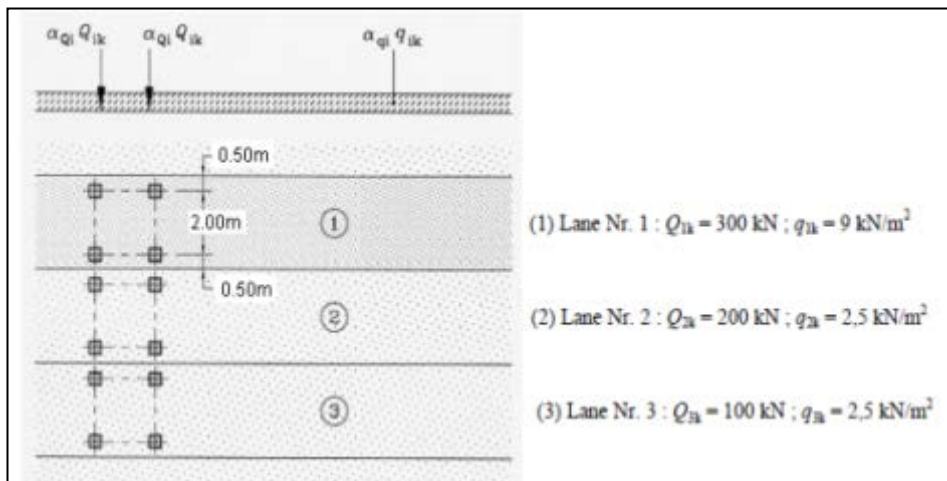
Load model 2

Load model 2 consists of a single axle load applied on a specific tyre contact area, particularly for the loading of shorter span bridges. The traffic loading model includes dynamic amplification within the models.

The axle load is defined as: $\beta_Q Q_{ak}$

where:

β_Q is the adjustment factor defined by the national annex or taken as 1.0.

Q_k is equal to 400 kN

The contact area of each wheel is defined as 350mm x 600mm which gives a maximum pressure of ($\beta_Q x$952kPa).

The axle and wheel contact area arrangement is shown in figure A.26.

Figure A.26   Load model 2
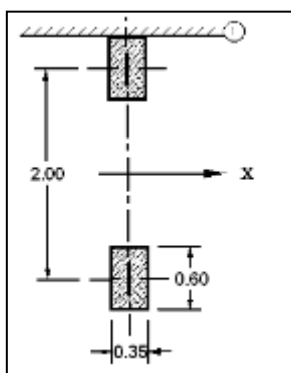
Load model 3

Load model 3 is defined as special vehicles and includes a set of assemblies of axle loads which are intended to represent the effects of abnormal vehicles which do not comply with the national regulations concerning weights and dimensions of normal vehicles. Dynamic amplification is not included in the vehicle models of load model 3 and is applied separately as per section A8.1.4.

The vehicles range from a total weight of 600kN with four axles to a total weight of 3600kN with 18 axles.

The axle loads range from 100kN, 150kN, 200kN and 240kN.

The axle spacing is typically 1.5m with some vehicles splitting the axle sets with a spacing of 12m.

The axle width is defined as 3m wide for axle loads up to 200kN and takes up one notional lane.

The axle width is defined as 4.5m wide for axle loads of 240kN and takes up two notional lanes.

If the load model is defined to move at low speeds then the dynamic amplification does not apply. Each lane is loaded by load model 1 and for the lane loaded with the special vehicle, the load model 1 loading is applied, with frequent values ($\Psi_1\ factors$), as shown in table A.28, included, but not in the region of 25m in front and behind the outer axles of the special vehicle.

If the load model is defined to move at normal speeds then a pair of special vehicles is used in the lane(s) occupied by these vehicles and the remaining area is loaded with load model 1, with frequent values ($\Psi_1\ factors$) included. In addition, dynamic amplification is applied by amplifying the special vehicle loading by the dynamic amplification factor in section A8.1.4.

Load model 4

Load model 4 relates to crowd loading and is not relevant for discussion for this project.

A8.1.3   Lane width and number of lanes

The design lane width is typically 3m.

The number of lanes is typically defined as the integer of $w/3$ where w is the width of the bridge.

The lane width and number of lanes may vary for bridges less than 6m and the number of lanes and lane width can be determined from table A.25.

Table A.25   Number of lanes

Carriageway width, w	Number of notional lanes, n	Width of a notional lane	Width of the remaining area
W < 5.4m	1	3m	w – 3m
5.4m ≤ w < 6m	2	$w/2$	0
6m ≤ w	$w/3$	3m	w – (3 x n)

A8.1.4   Dynamic load allowance

The DLA is integrated within the loading models except for load model 3 where the dynamic allowance factor is calculated by the following equation:

$$\varphi = 1.40 - \frac{L}{500}, \qquad \varphi \geq 1$$

where:

   L is the influence length.

A8.1.5   Horizontal loads

Braking forces

The braking force is taken as a proportion of one lane of loading of load model 1 and is determined by the following equation:

$$Q_{lk} = 0.6\alpha_Q(2Q_k) + 0.10\alpha_Q q_k wL$$

$$180\alpha_Q(kN) \leq Q_{lk} \leq 900(kN)$$

This is more simply described as: braking force = 60% total axle loads + 10% uniformly distributed load.

The braking force is applied at the deck surface.

Centrifugal forces

The centrifugal force, Q_{tk} is taken as a proportion of the maximum weight of the tandem vehicles of load model 1 and is defined by the following equations in table A.26 below:

Table A.26   Centrifugal force

$Q_{tk} = 0.2Q_v$ (kN)	If r < 200m
$Q_{tk} = 40Q_v/r$ (kN)	If 200m $\leq r \leq$ 1500m
$Q_{tk} = 0$	If r > 1500m

where: r is the horizontal radius (m)

Q_v is the total maximum weight of the vertical concentrated loads of the tandem systems of load model 1 for one or all lanes loaded.

The centrifugal force is applied at the deck surface.

A8.1.6   Load factors

To evaluate the design effects, EN 1990 specifies the combinations of loads and the partial load factors, γ, and combination factors, Ψ, to be applied to these loads. The partial load factors are applied to all loads at the ULS to increase the design action to take into account the possibility of unfavourable deviation of the design action. Partial load factors are not included at the SLS. The combination factors are applied, typically where more than one variable action is considered, to take into account the reduced probability of these variable actions occurring simultaneously and is typically applied to the non-critical additional variable action(s).

Ultimate limit state

For the ULS the format for calculating the design effects for a combination of actions is defined as:

$$E_d = \sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i \geq 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$$

or

$$E_d = \sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} \Psi_{0,i} Q_{k,1} + \sum_{i \geq 1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$$

where: $\gamma_{G,j}, \gamma_P, \gamma_{Q,1}$ are the partial factors for permanent, prestressing and variable actions

$G_{k,j}, P, Q_{k,1}$ are the permanent, prestressing and variable action design actions

$\Psi_{0,i}$ is the factor for combination value of a variable action.

The following partial load factors in table A.27 are specified in EN 1990 annex A2.

Table A.27   Partial load factors

Action	Partial load factor
Traffic live load	1.35
Concrete self weight	1.35
Steel self weight	1.35
Superimposed dead load	1.35
Surfacing	1.35
Wind	1.50

Serviceability limit state

For the SLS the format for calculating the design effects for a combination of actions for the following serviceability limit states are:

1   Characteristic combination – irreversible limit states

$$E_d = \sum_{j\geq1} G_{k,j} + P + Q_{k,1} + \sum_{i\geq1} \Psi_{0,i} Q_{k,i}$$

2   Frequent combination – reversible limit states

$$E_d = \sum_{j\geq1} G_{k,j} + P + \Psi_{1,1} Q_{k,1} + \sum_{i\geq1} \Psi_{2,i} Q_{k,i}$$

3   Quasi-permanent – long-term effects, appearance

$$E_d = \sum_{j\geq1} G_{k,j} + P + \sum_{i\geq1} \Psi_{2,i} Q_{k,i}$$

where: $G_{k,j}, P, Q_{k,1}$ are the permanent, prestressing and variable action design actions

Ψ_0 is the factor for combination value of a variable action

$\Psi_{1,}$ is the factor for frequent value of a variable action

Ψ_2 is the factor for quasi-permanent value of a variable.

For the normal traffic loading conditions, there is no partial load factor applied at the SLS and thus for comparison purposes the partial load factor for live load can be considered to be unity or 1.0.

The factors $\Psi_0, \Psi_{1,}, \Psi_2$ for the combination, frequent or quasi-permanent values are applied to represent the probability of the combination of variable actions for the different limit states and are shown in table A.28.

Table A.28   Recommended values of Ψ factors for road bridges

Action		Factor
Traffic combination factor ψ_0	Tandem system	0.75
	Uniformly distributed load	0.40
Wind combination factor ψ_0		0.60
Traffic combination factor ψ_1	Tandem system	0.75
	Uniformly distributed load	0.40
Traffic combination factor ψ_2	Tandem system	0.00
	Uniformly distributed load	0.00

A8.1.7   Evaluation of bridges

The scope of the Eurocodes does not extend to the evaluation or assessment of existing bridges. It is noted that there is a long term aim to develop a Eurocode for assessment. The evaluation of bridges is therefore carried out as per the existing assessment guidelines or standards for the reference country. These assessment standards are not discussed here further except for the UK where the existing standard BD 21/01 is discussed in section A9.1.7.

A8.1.8   Background

Work on the definition of traffic loads on road bridges started in Europe in 1987. As reported by Prat (2002), various universities (Aachen, Delft, Trinity College, Dublin, Liege, Pisa), consulting engineers (Flint & Neill Partnership) and organisations such as Transport Research Laboratory and British Research

Establishment in the UK together with SETRA and LCPC in France led studies involving civil engineering companies and industrial manufacturers of heavy vehicles and tyres for the determination of loading levels for bridge design. It was decided that the load model should consist of a normal traffic load model calibrated to the effects of measured traffic data and a classified abnormal traffic load model that might be chosen in case exceptional vehicles not covered by the normal traffic load model were foreseen. It was accepted that the normal load model should also be composed of concentrated loads and uniformly distributed loads in such a way that it would be suitable for both global and local analyses of the bridge in both the longitudinal and transverse directions and be suitable for ULS and SLS.

Road traffic actions in EN 1991-2 are represented by a series of load models that represent different traffic situations and different components of traffic action (eg horizontal force).

Load model 1 consists of concentrated (double axle tandem system) and uniformly distributed loads which cover most of the effects of truck and car traffic. As the code specifies the live load to be used in each traffic lane, there is no need to introduce multi-lane factors.

Load model 2 consists of a single axle load of 400kN, which covers the dynamic effects of normal traffic on short structural members.

Load model 3 consists of sets of axle loads representing special heavy load vehicles, which can travel on routes permitted for abnormal loads.

Load model 4 represents crowd loading of 5kPa intended for bridges in or near urban areas.

Load models 1 and 2 are deemed to represent the most severe traffic met or expected in practice, other than that of special vehicles requiring permits to travel, on the main routes of European countries. Load models 1 and 2 incorporate adjustment factors α and β respectively which are intended to be set in the national annexes that accompany the core document.

The development and subsequent calibration of the load models 1 and 2 is described in Bruls et al (1996a), Flint and Jacob (1996), Bruls et al (1996b) and Calgaro and Sedlacek (1992).

The calibration was based upon a comparison of WiM data across several countries (particularly France, Germany, Italy and Spain) which included information on axle weights of heavy vehicles, spacing between axles and between vehicles and length of vehicles. The data recorded on the A6 motorway near Auxerre, France was chosen to define the European traffic load model. The data comprised 2 x 2 lanes and was considered to represent a future trend in traffic development on other roads in view of the percentage of articulated heavy vehicles, the load rates and the weights. Dynamic effects were investigated at the site and subsequently filtered and removed from the statistical distributions to obtain purely static data from which random vehicle sequences were formed using Monte Carlo methods to take account of the variability of the traffic effects. Loading scenarios for flowing (at different speeds) and congested (slowly moving) traffic situations were identified for a large variety of simply supported and continuous bridges with different superstructure cross sections, different widths and different span lengths. These were used to calculate the maximum action effects (bending moments, shear forces) for establishing distribution curves for these maximum effects.

EN1991-2 states that the characteristic target values of the traffic effects were determined for a 1000 year return interval (probability of 5% exceedance in 50 years) and that frequent or mean values corresponded to a return interval of one week on main roads in Europe. Target values for between one and four loaded lanes were determined. Load model 1 was then developed and calibrated against the target values.

Prat (2002) reported on two reassessments of the main load model 1 in 1997 (France) and 2001(Belgium) in relation to more recent WiM traffic data. Significant advances had been made in improving the accuracy

of WiM systems and there was interest in whether the traffic patterns had changed in the 10 years since the initial calibration work had been undertaken. Both studies confirmed the original calibration work for load model 1.

Recent work by O'Brien et al (2008) looked at the possible introduction in Europe of longer and heavier trucks with up to eight axles and gross weights of up to 60t, associated with reducing the number of vehicles for a given volume or mass of freight, and the concerns of road authorities on the impact on Europe's bridge infrastructure. The conclusions from this study were that there is considerable conservatism in the Eurocode traffic loading model and that bridges designed to this or similar codes of practice can be shown to be safe in the presence of significant numbers of longer and heavier trucks. The paper demonstrated the importance of cranes and low-loaders as the dominant feature in extreme traffic loading in bridges at a very heavily trafficked site in the Netherlands. Medium wheelbase non-permit trucks did not appear to make a great contribution.

A9   United Kingdom

A9.1   National annex to British standard BS EN 1991-2

A9.1.1   General

The *National annex to British standard BS EN 1991-2* (BSI 2008) is the UK implementation of *EN 1991-2:2003 Traffic loads on bridges*. Primarily this document provides the α-factors for load model 1 and the load combination factors ψ. The report by Atkins (2005) discusses the calibration of the load adjustment factor, α, against the UK Highways Agency's (2001) standard BD 37/01 and is discussed further in section A9.1.8.

A9.1.2   Traffic loading

Following the Eurocode format, the UK *National annex to BS EN 1991-2* consists of four load models, modified accordingly and where allowable, to cover the traffic situations to be taken into account for design of bridges in the UK.

Load model 1

Based on calibration studies, the α-factors for the UK *National annex to BS EN 1991-2* are summarised in table A.29:

Table A.29   α-factors

Lane	EN 1991-2		UK National annex to BS EN 1991-2	
	Tandem axles	Uniformly distributed load	Tandem axles	Uniformly distributed load
Lane 1	1.0	1.0	1.0	0.6111
Lane 2	1.0	1.0	1.0	2.2
Lane 3	1.0	1.0	1.0	2.2
Other	-	1.0	-	2.2
Remaining area		1.0	-	2.2

Based on the factors in table A.29 the following loading values shown in table A.30 for the axle loads and uniformly distributed loads for load model 1 are applied. The application of load model 1 is as per EN 1991-2 as shown in section A9.1.2.

Table A.30   Load model 1: loading values

Location	Tandem system TS	Uniformly distributed load system
	Axle loads Q_k (kN)	q_k (kN/m²)
Lane number 1	300	5.5
Lane number 2	200	5.5
Lane number 3	100	5.5
Other lanes	0	5.5
Remaining area	0	5.5

Load model 2

The β factor in the UK *National annex to BS EN 1991-2* is specified as 1.0 which equates to an axle load of 400kN.

The wheel contact area is defined as 400mm x 400mm which gives a maximum pressure of 1250kPa.

The application of load model 2 is as per EN 1991-2 as shown in section A8.1.2.

Load model 3

Load model 3 is defined as special vehicles and the UK *National annex to BS EN 1991-2* specifies three special vehicles (SVs) to represent the maximum design effects that could be induced by actual vehicles in accordance with the special types general order (STGO) regulations. The UK *National annex to BS EN 1991-2* also specifies four special order vehicles (SOVs) to represent the maximum design effects that could be induced by actual vehicles in accordance with the special order (SO) regulations.

The following three SV vehicles simulate the vertical effects of STGO vehicles and the arrangements are shown in figure A.27:

1   SV80 vehicle

   a   Maximum weight is 80t.

   b   Contains two sets of tri-axle sets with an axle load of 130kN.

   c   The axle spacing within the group is 1.2m and the spacing between the tri-axle sets is variable and taken as the critical of 1.2m, 5.0m or 9.0m.

2   SV100 vehicle

   a   Maximum weight is 100t.

   b   Contains two sets of tri-axle sets with an axle load of 165kN.

   c   The axle spacing within the group is 1.2m and the spacing between the tri-axle sets is variable and taken as the critical of 1.2m, 5.0m or 9.0m.

3   SV196 vehicle

   a   Maximum weight is 196t.

   b   Contains four multi-axle sets with a maximum axle load of 180kN, the axle sets and loads are shown in figure A.27.

   c   The axle spacing within the group is typically 1.2m but also varies.

The following four SOV vehicles simulate the vertical effects of SO vehicles. The standard configuration of the SOV model consists of a trailer with two bogies to support the payload weight and two tractors, one pushing and one pulling the trailer. If the gradient of the road is steeper than 1:25 then six tractor units are used in any combination. The arrangements of the vehicles are shown in figure A8.2.

1   SOV250 vehicle

    a   Maximum weight of trailer is 250t.

    b   Contains a six-axle bogie and five-axle bogie of the trailer with axle loads of 225kN.

2   SOV350 vehicle

    a   Maximum weight of trailer is 350t.

    b   Contains two eight-axle bogies of the trailer with axle loads of 225kN.

3   SOV450 vehicle

    a   Maximum weight of trailer is 450t.

    b   Contains two 10 axle bogies of the trailer with axle loads of 225kN.

4   SOV600 vehicle

    a   Maximum weight of trailer is 600t.

    b   Contains a 14-axle bogie and 13-axle bogie of the trailer with axle loads of 225kN.

The axle spacing within the bogie groups is 1.5m and the spacing between bogies is variable from 1.5m to 40m. The spacing between the tractor and bogie is 5.0m. The tractor weighs a maximum of 6t and has a 100kN front axle and a tri-axle set with 165kN axle loads. The axle spacing of the tractor is 1.85m from front axle and 1.35m within the tri-axle set.

Figure A.27 Load model 3 – SV vehicles
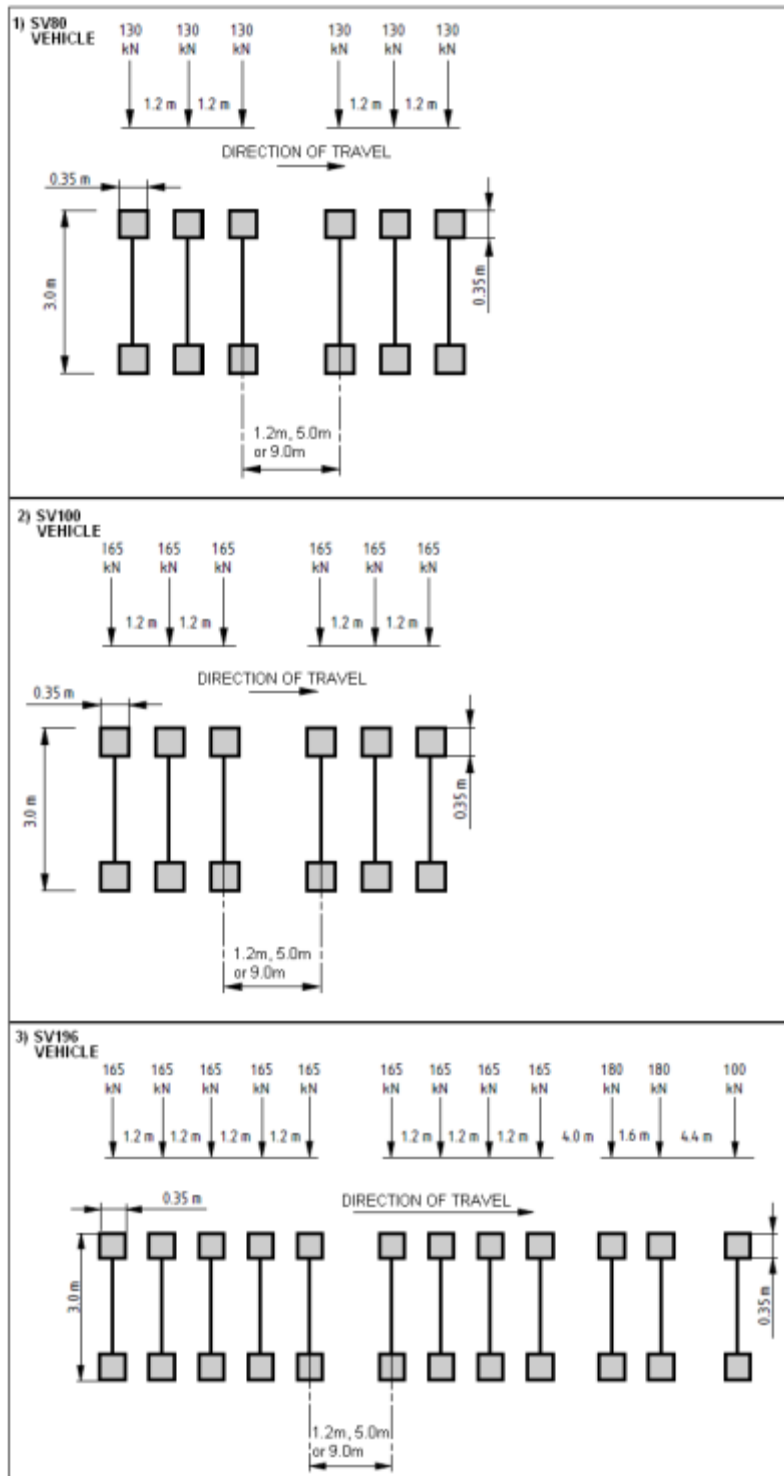
Figure A.28   Load model 3 – SOV vehicles
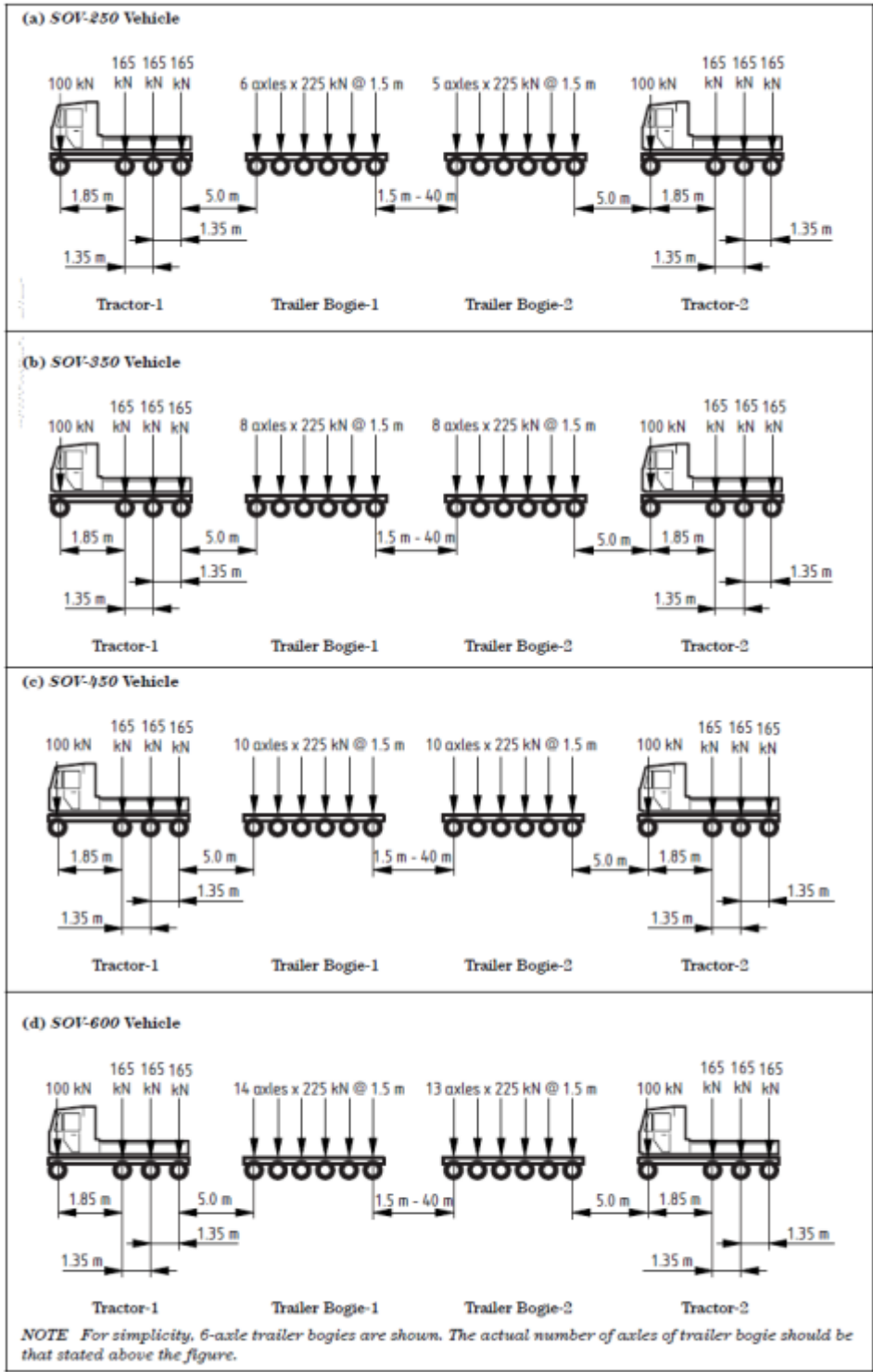
Bridges with load model 3 only have one SV or SOV vehicle loaded and placed transversely within one notional lane or straddling two adjacent lanes. The remaining area of the bridge is loaded with load model 1 with frequent values, Ψ, applied to the loading. For the lane loaded with the SV, the load model 1 loading is also applied but not in the region of 5m in front and behind the outer axles of the SV.

The axle loads in load model 3 do not include dynamic effects and the dynamic amplification factor in section A9.1.4 is applied depending on the magnitude of the basic axle load:

Load model 4

Load model 4 relates to crowd loading and is not relevant for discussion for this project.

A9.1.3   Lane width and number of lanes

The design lane width and number of lanes is as per EN 1991-2 as shown in section A8.1.3.

A9.1.4   Dynamic load allowance

The DLA is integrated within the loading models except for load model 3 where the dynamic amplification factor is determined from table A8.3 below depending on the basic axle load.

Table A.31   Dynamic amplification factor

Basic axle load	Dynamic amplification factor
100kN	1.20
130kN	1.16
165kN	1.12
180kN	1.10
225kN	1.07

A9.1.5   Horizontal loads

Braking forces – normal loading

The braking force for normal loading is as per EN 1991-2 as shown in section A8.1.5.

Braking forces – load model 3

For load model 3, the longitudinal force is taken as the more severe of braking or acceleration forces defined by the following:

Braking force (per axle) $Q_{lk,S} = \delta w$

Where: w is the basic axle load of the SV or SOV vehicle

δ is taken from table A.32.

Table A.32 δ factors

Vehicle	δ
SV80	0.5
SV100	0.4
SV196	0.25
SOV vehicles	0.2

The acceleration force is taken as 10% of the gross weight of the SV or SOV vehicle and distributed between the axles in the same proportion as the vertical loading.

Centrifugal forces – normal loading

The centrifugal force, Q_{tk}, for normal loading is as per EN 1991-2 as shown in section A8.1.5.

Centrifugal forces – load model 3

The centrifugal force, $Q_{tk,S}$, from the SV or SOV vehicles is taken as the following:

$$Q_{tk,S} = \frac{W \; x \; V^2}{g \; x \; r}$$

where: W is the weight of the SV or SOV vehicle (kN)

V is the velocity of the SV or SOV vehicle (m/s) $\leq V_{limit}$

V_{limit} is the speed limit on the road (m/s)

g = acceleration due to gravity (9.8m/s²)

r is the radius of curvature (m)

V is calculated by the following equation:

$$V = \rho \left\{ greater \; of \; 30 \; or \; \sqrt{\tfrac{100xgxr}{r+150}} \right\}$$

ρ is taken from table A8.5

Table A.33   ρ factors

Vehicle	ρ
SV80	0.86
SV100	0.77
SV196	0.55
SOV250	0.41
SOV350	0.36
SOV450	0.33
SOV600	0.3

The centrifugal force is applied at the deck surface.

A9.1.6   Load factors

Ultimate limit state

For the ULS the format for calculating the design effects for a combination of actions is defined as per EN 1990 and is shown in section A8.1.6.

The following partial load factors in table A.34 are specified in the UK National Annex to EN 1990 annex A2 (BSI 2009). The partial load factors specified in EN 1990 annex A2 are shown for comparison purposes.

Table A.34   Partial load factors

Action	UK National annex to BS EN 1990 annex A2	EN1990:2002
	Partial factor	Partial factor
Traffic live load	1.35	1.35
Concrete self weight	1.35	1.35
Steel self weight	1.20	1.35
Superimposed dead load	1.20	1.35
Surfacing	1.20	1.35
Wind	1.70	1.50

Serviceability limit state

For the SLS the format for calculating the design effects for a combination of actions is defined as per EN 1990 and is shown in section A8.1.6.

For the normal traffic loading conditions, there is no partial load factor applied and thus for comparison purposes the partial load factor for the live load can be considered to be 1.0.

The combination, frequent and quasi-permanent factors for the UK *National annex to EN 1990 annex A2* are summarised in table A.35. The factors specified in EN 1990 annex A2 are shown for comparison purposes.

Table A.35   ψ-factors

Action		UK National annex to BS EN 1990 annex A2	EN1990:2002
		Factor	Factor
Traffic combination factor ψ_0	Tandem system	0.75	0.75
	Uniformly distributed load	0.75	0.4
Wind combination factor ψ_0		0.50	0.6
Traffic combination factor ψ_1	Tandem system	0.75	0.75
	Uniformly distributed load	0.75	0.4
Traffic combination factor ψ_2	Tandem system	0.00	0.00
	Uniformly distributed load	0.00	0.00

A9.1.7   Evaluation

As the scope of the Eurocode does not extend to assessment of bridges in Europe and in particular to the UK, the standard *Design manual for roads and bridges* (DMRB) *BD 21/01 The assessment of highway bridges and structures* (BD 21/01) (Highways Agency 2001b) is used to evaluate bridges. The scope of the standard is intended to cover bridges built prior to 1922 and bridges built after 1922 which were not designed for HB30 loading. The standard covers the assessment of bridges constructed of steel, concrete, wrought iron or cast iron. The standard adopts the limit state format with partial safety factors and predominately assesses bridges for HA loading up to 40t.

The assessment procedure follows the following equation to determine the adequacy of the structure to carry an assessment load. In simplified terms, the equation assesses whether the design actions for the load combination are less than the resistance or structural capacity.

$$R_A^* \geq S_A^*$$

where: R_A^* is the assessment resistance (including material strength partial factors)

S_A^* is the assessment load effect (including partial load factors).

The assessment load effects include dead loads, superimposed dead loads and live loads.

If the equation above is not satisfied, consideration is given to weight and/or lane restrictions and repair, strengthening or reconstruction of the structure.

Structures which cannot sustain the 40t assessment live loading and are not scheduled for replacement or strengthening are reassessed for the following reduced live loads to determine load restrictions on the bridge and/or consideration for closure:

• 26t live load

- 18t live load

- 7.5t live load

- fire engine live loading (vehicle range of 6.6t to 16.26t)

- 3t live loading (cars and vans).

When the structure cannot sustain any of these loadings, it should be immediately closed.

A9.1.8 Background

Prior to issue of EN 1991-2, traffic live loads were specified in British standard BS 5400 part 2 (BSI 2006). UK standard *Design manual for roads and bridges* (DMRB) *BD 37/01 Loads for highway bridges* (BD 37/01) (Highways Agency 2001) specified the design traffic loadings for highway bridges and associated structures, these being an adaption of the BS 5400 requirements. All road bridges were required to be designed to carry HA loading. In addition, a minimum of 30 units of type HB loading was also to be considered for all road bridges. The actual number of units was related to the class of road with major roads designed for a maximum of 45 units HB. HA loading comprised a uniformly distributed load (the intensity of which varied with loaded length) and either a constant knife-edge load or a single wheel and represented normal traffic. Lane factors applied for different lanes accounted for coexistent loading in adjacent lanes as a function of loaded length. HB loading was an abnormal vehicle unit load comprising two groups of twin axles with variable inner axle spacing.

The type HA load model was first introduced in 1988 and included the following considerations:

- For spans up to 50m it was derived using a deterministic method by evaluating the extreme load effects caused by single, tandem and multiple vehicles over bridge influence lines and fitting a uniformly distributed and knife-edge load model to envelope these effects.

- The loading for spans greater than 50m was derived from probabilistic simulation of extreme traffic effects using a Monte Carlo simulation (Atkins 2005).

- An overloading factor of 1.4 was included based on vehicle weights from roadside surveys using a static weigh bridge. The overload factor was reduced linearly from 1.4 at 10m to 1.0 at 60m loaded length.

- An impact factor of 1.8 was included in the maximum axle load of the single vehicle case.

- A bunching factor was included to account for the possible increase in loading caused by three vehicles bunching side by side into two lanes of 3.65m standard width.

- An allowance of 10% was made for future increases in loading.

For long spans, traffic flow rates, percentage of heavy vehicles in the flow, frequency and duration of queuing traffic and the spacing of vehicles in the queues were considered. The effect of these parameters was assessed by studying traffic patterns and by load surveys. Statistical methods were used to derive characteristic loads and then nominal loads.

In preparing the UK *National annex to EN 1991-2*, the basis of the HA short-span loading model was reviewed in light of increased legal limits of vehicle weights, levels of overloading observed from WiM data and the dynamic effects of traffic loading on bridges (Atkins 2005). The deterministic procedure used for the derivation of the type HA loading model given in BD 37/01 for short spans was repeated with revised values for vehicle and axle loads, overload factor and dynamic amplification factor. The effect of bunching was removed. The results of this study confirmed that BD 37/01 was a reasonable basis for use in calibration of the α load adjustment factors for implementing EN 1991-2 in the UK.

The Eurocode load model LM1 is very different from the type HA loading model given in BD 37/01 in terms of load configuration, load intensities and the way the carriageway is divided into notional lanes.

Atkins (2005) describes how these differences were considered in determining the appropriate α-factors by deterministically fitting the nominal and design load effects to those resulting from BD 37/01. Atkins (2005) further outlines the process followed in determining the partial load factors adopted for the ULS. A target reliability index of 5.8 was chosen for the calibration of partial load factors.

The wheel loading of 200kN specified by the UK *National annex to BS EN 1991-2* is an increase to the 100kN load specified in BS 5400-2. However the published document PD6688-2 (BSI 2001) states that the wheel contact area has been adjusted to result in similar contact pressures to those previously given in BS 5400-2.

A9.2   Interim advice note 124/11: Use of Eurocodes for the design of highway structures

The Highways Agency (2011b) *Interim advice note 124/11 Use of Eurocodes for the design of highway structures* provides guidance and requirements for the use of Eurocodes for the design of highway structures (including geotechnical works) on the English strategic road network. The advice note does not apply to the assessment of existing structures.

The advice note lists the sections of the DMRB applicable for use with the Eurocodes and national annexes and accompanying British standards published documents.

The advice note states the following:

- Highway structures that carry traffic loads must be designed for BS EN 1991-2 load models 1, 2 and, if appropriate for pedestrians, 4.

- Highway structures that carry motorway and trunk road traffic must also be designed for all the SV models given in load model 3 as defined in the UK *National annex to BS EN 1991-2*.

- Where a highway structure carries traffic on a route that is designated by the overseeing organisation as a heavy load route, the structure must be designed for the appropriate SOV models as defined in the UK National Annex to BS EN 1991-2 and/or specific individual vehicles as required. These loads must be agreed with the technical approval authority.

A9.3   BD 21/01: The assessment of highway bridges and structures

The UK standard *DMRB BD 21/01 The assessment of highway bridges and structures* (BD 21/01) (Highways Agency (2001b) nominates the requirements for the assessment of highway bridges for vehicles complying with the legal weight limits using limit states principles. The ULS is used together with appropriate partial load factors.

BD 21/01 provides detailed factors to be applied to the live load, which depend on the quality of the road surface and the volume of truck traffic. The carrying capacity is normally assessed relative to the loading possible from any convoy of vehicles of up to 40/44t gross vehicle weight – an adjusted type HA loading.

The values of the assessment live loadings were determined in a similar way to the type HA loading but using an envelope containing those vehicles whose gross weight was equal to or less than the maximum weight specified for the particular loading. No 10% contingency allowance was included in the calculations.

Research carried out as part of the derivation of the revised short-span assessment load in BD 21/01 in 1997 showed that free-flow traffic with high-speed impact and without lateral bunching was the most

onerous condition for short-span bridges up to 50m loaded length. Hence the bunching factor was subsequently removed in deriving the assessment live load in BD 21/01.

A review of the assessment loading indicates that the 95% characteristic load (5% probability of exceedance in 120 years) was approximately the same as the 1.2 x HA loading. The ultimate load (1.5 x HA) had an equivalent return period of 200,000 years or 0.06% chance of exceedance in 120 years.

A9.4 BD 86/11: The assessment of highway bridges and structures for the effects of special types general order (STGO) and special order (SO) vehicles

DMRB BD 86/11 The assessment of highway bridges and structures for the effects of special types general order (STGO) and special order (SO) vehicles (BD 86/11) (Highways Agency 2011a) is a companion document to BD 21/01 relating to the assessment of highway bridges and structures for the effects of STGO and special order (SO) vehicles – abnormal vehicles. The ULS is used together with appropriate partial load factors.

Previously, highway bridges and structures were assessed using the HB vehicle to determine their capacity for carrying abnormal vehicles (ie those which are subject to notification or permit requirements). The model was the same as that used for the design of new structures. A number of studies showed that the HB model did not represent accurately the load effects induced by real abnormal vehicles. In particular, because of the high axle weights, the HB model was found to be excessively conservative for very short-span structures, although this conservatism reduced for spans between 15m and 30m. Further it was shown that real abnormal vehicles could produce more severe load effects than an HB model vehicle of the same gross weight Atkins (2002). This was because the real STGO vehicles often had more axles and smaller axle spacing than those of the HB model vehicle.

The Highways Agency thus embarked on a study to develop a new loading model which represented the effects of abnormal vehicles more realistically than the HB model for use with BD 86/11. Atkins (2002) describes the process followed in reviewing data from actual permit vehicles, fictitious vehicles which complied with the overload regulations and WiM data to develop a SV model to represent the effects of abnormal vehicles.

Five SV vehicles were developed with gross weights and axle spacings which enveloped the critical load effects from real STGO vehicles. Overload factor and dynamic amplification factor values were developed so that they could be modified where the STGO vehicle transit was better regulated and where there was a greater confidence on vehicle weights.

Load model 3 in the UK *National annex to BS EN 1991*-2 includes three SV vehicles, two of them (SV80, SV100) are identical to those in BD 86/11 and are shown in figure A8.1. The SV196 vehicle in figure A.27 is similar to the SV train vehicle in BD 86/11 although the SV196 includes heavier axle loads. Load model 3 also includes the SOV vehicles, shown in figure A.28, which are identical to those in BD 86/11.

A10 Discussion and conclusions

A10.1 Economic summary and conclusions

There is a significant variation in gross vehicle weight limits between countries for equivalent classes of heavy vehicles and New Zealand tends to be at the lower end for both gross and axle weight limits.

Studies of proposed increases in heavy vehicle mass limits in Australia and New Zealand have consistently found that the benefits of allowing heavier trucks outweigh the costs and hence that limits should be increased. However the studies have almost always only considered modest increases in vehicle mass limits.

Austroads commissioned a project specifically to investigate the economics of higher bridge design loadings. The project aimed to look well beyond previous incremental increases in legal loading to try to find an end point in the evolution of the Australian general freight vehicle. The project found that optimal gross and axle mass limits were almost double existing mass limits. The benefits are very large – even in 1994/95 dollars and with the lower freight volumes being transported at that time benefits were up to $1 billion per annum – far greater than any incremental bridge and pavement costs.

Key findings in relation to bridge construction costs included:

- Bridge construction costs are not highly sensitive to increases in design vehicle mass, increasing approximately 1% for each 10t increase in axle set loading.

- Bridge construction costs also vary with the configuration of the vehicle, thus moving from a semi trailer as the design vehicle, to a B-double, and then to a B-triple, results in further increases of around 0.5% at each step.

- Bridge construction costs also vary slightly with the length of the design vehicle.

Corresponding percentages would need to be estimated or verified for New Zealand and applied to annual new bridge and bridge replacement budgets to determine the likely increase in costs associated with alternative bridge design loads.

The following are examples of the vehicle configurations and loads that were identified by the project as being the most appropriate basis for a new bridge design load standard in Australia.

Table A.36   Vehicle configurations used for the development of the bridge design load standard in Australia

Vehicle	Length (m)	GVM (tonnes)	Axle set loadings (tonnes)		
			Steer	Dual	Tri-axle
Articulated semi-trailer	14.8	73	7	26.5	39.5
B-double	21.4	112.5	7	26.5	39.5

For the following reasons these vehicles are likely to represent an **upper bound** of the potential future vehicle loading that should be adopted in determining the new bridge design load standard in New Zealand.

Benefits from New Zealand studies into increased mass limits have tended to be much less than those obtained in the Australian studies. Possible reasons for this include longer freight haul distances and less mountainous terrain in Australia, a smaller freight task and fewer trucks in New Zealand and a smaller proposed increase in vehicle loads in the studies reviewed.

New Zealand's road user charges (RUC) system is also likely to have an influence on future heavy vehicles. RUC rates increase exponentially for heavier vehicles. This is likely to diminish the attractiveness to transport operators of moving to the much heavier trucks and axle loads operated in other countries.

These and other factors mean that the optimal upper limit of vehicle loads is likely to be lower for New Zealand than for most other developed countries, at least for as-of-right higher limits on all roads. However if the incremental cost of building stronger bridges is small it may be reasonable to calibrate the design loading to lean towards the upper bound rather than the lower bound if there is uncertainty about the most economic design load.

The original VDAM Rule Amendment proposal included an option of an across-the-board increase in gross mass to 50t from the current 44t gross mass limit and an increase in permitted vehicle length to 22m from 20m. This option was removed from the final VDAM Rule Amendment 2010 but it should be noted that both the Transit NZ Heavy Vehicle Limits project and studies for the VDAM Rule Amendment 2010 showed it to be economically justified. It should therefore be regarded as the **lower bound** of potential future vehicles for a new bridge design load.

There were over 1000 applications for HPMV permits under the VDAM Rule Amendment 2010, indicating a large demand for higher mass limits. The applications could provide a valuable indicator of the direction in which vehicle types and mass limit increases might go and hence that should be considered for future bridge design loadings.

Australian government road freight projections to 2030 assume that heavy vehicle productivity will improve by 2% per annum for inter-regional road freight, and 1% per annum for shorter-distance intra-regional road freight in line with trends in road freight vehicle average loads observed over the last 10 to 15 years. On this basis, heavy vehicle loads could be expected to increase by up to 40% over the next 20 to 30 years.

Another issue in relation to increasing the design loading is whether the anticipated higher mass limits could feasibly be accommodated on the road system, or whether the capital expenditure required to upgrade the existing infrastructure would preclude this eventuality within the foreseeable future. Such an assessment is outside the scope of this project but the issue should be considered. Factors to consider would include the appropriate discount rate and the average remaining life of existing bridges.

These economic drivers should be considered in conjunction with the approach of determining the bridge design loading based on current WiM data.

A10.2 Traffic load models

There are a number of different model representations for the design traffic load in the international standards reviewed:

- The Eurocode uses a double axle tandem and a uniformly distributed load

- The AASHTO code uses a design truck or double axle tandem in combination with a uniformly distributed load.

- The AS5100 uses two load models each comprising four tri-axles in combination with a uniformly distributed load. The code also includes a separate heavy axle or wheel loading.

- The CAN/CSA-S6-06 uses a design truck or double axle tandem in combination with a uniformly distributed load.

With the exception of the Australian standard, the development of the traffic load model has been based on review of WiM data and choice of a configuration of the load model to produce similar load effects to that of measured vehicles. Variability of the design load (and hence choice of load factor) has been determined based on traffic survey data and historical trends in vehicle weights and truck numbers, with consideration of likely variance over the design life of the structure. In the Australian case, the review of WiM data indicated that the T44 loading was insufficient and the development of the T90 and T55 vehicles (Heywood 1995a) was a better representation of the traffic loading based on the WiM data. However, to account for future increases in loading and to prevent limitations to productivity, a bridge design loading standard was developed to provide a reasonable representation of future vehicles and resulted in a much heavier design loading.

The traffic loading models developed are designed to accommodate the loading conditions for various span lengths. For shorter-span bridges, vehicles may only have one axle or an axle set loaded on the

bridge which may include an individual overloaded axle or axle set of the vehicle. For medium-span bridges, the full vehicle length is loaded on the bridge and for long-span bridges a series of bunched vehicles may be loaded on the bridge. The design traffic loading has to accommodate all these effects and the international design codes either separate these loadings into separate entities or are accommodated within the traffic loading model.

Short spans and local effects

The design effects for short spans and local elements are typically governed by wheel or axle loading. The international design codes address this loading arrangement as follows:

*   For the AS5100, the loading for the shorter-span bridges and local effects are governed by the 80kN wheel load or the 160kN axle load. The contact area results in a maximum pressure of 1120kPa (including dynamic effects).

*   The AASHTO code, for a single lane of loading, uses an axle load of the truck of 170.8kN and also includes a closely spaced double axle tandem loading each with an axle load of 133.4kN.The maximum pressure from the wheel loading is 733kPa (including dynamic effects).

*   The CAN/CSA-S6-06 uses any individual axle of the design truck and the design truck includes an overloaded axle load of 175kN. The design truck also includes a closely spaced axle set. The maximum pressure from the wheel loading is 816kPa (including dynamic effects).

*   The Eurocode uses a single axle load of 400kN from load model 2. The maximum pressure from the wheel loading is 952kPa (dynamic effects built in).

*   The UK *National annex to BS EN 1991-2* uses a single axle load of 400kN from load model 2 with a reduced contact area compared with EN 1991-2. The maximum pressure from the wheel loading is 1250kPa (dynamic effects built in).

*   The *Bridge manual* uses an axle load of 120kN with an SLS factor of 1.35 for normal loading or an overloaded axle of 240kN. The maximum pressure of the wheel loading is 1050kPa (including dynamic effects).

Figure A.29 compares the wheel load pressure of the different design codes.

Figure A.29   Wheel load pressure
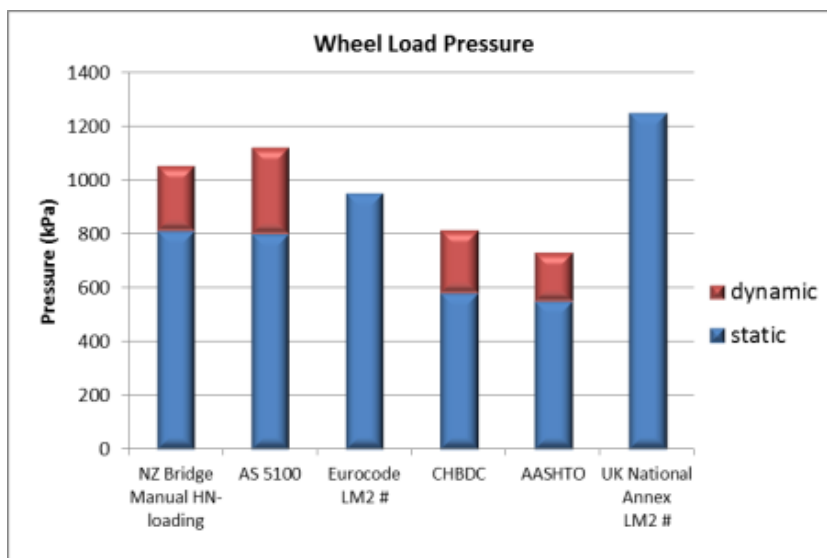
\# Dynamic effects already included in loading.

Figure A.30   Axle loads
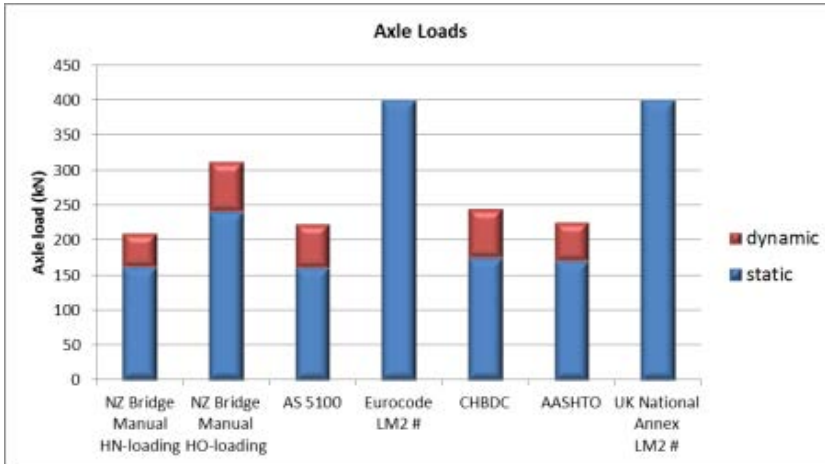
Dynamic effects already included in loading.

It can be seen from figure A.30 that the New Zealand axle loading (HN loading) is low in comparison with other international design codes.

Figure A.31   Comparison of SLS sagging moment for a single lane loaded (DLA inc)
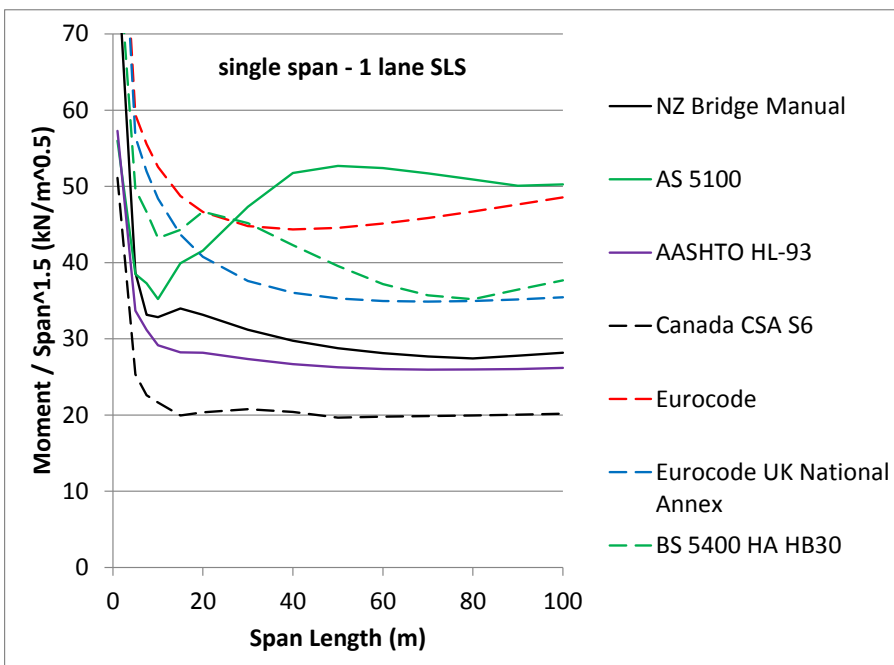
Medium-span bridge loading

For medium-span bridges, the full vehicle loading component is loaded on the bridge and accompanied by a traffic stream. As discussed above, the traffic loading for the design codes typically comprises a design vehicle, or series of axle loads, combined with a uniformly distributed load to represent the accompanied traffic stream. For medium-span bridges the design vehicle induces the highest component of loading, however as the span length increases, the queued traffic or the uniformly distributed load becomes the dominant component of loading.

A comparison of the bending moments for simply supported spans up to 100m for a single lane of loading is shown in figure A.31. The New Zealand loading is towards the middle of the range in terms of

magnitude of the load for bridges up to 50m. It is clear from this graph that the Australian standard, Eurocode and the UK national annexes include a higher loading compared with the other design codes.

In comparing the magnitude of the uniformly distributed loading component and the design vehicle component of the loading, there is no clear observation of any trends between the codes. In comparing the bending moments, it is clear that the AASHTO, CAN/CSA-S6-06, *Bridge manual* and the T44/L44 loadings follow a similar trend. This trend is representative of actual loading in the North American region. The New Zealand and T44/L44 loadings also follow a similar trend as these loadings were based on previous loading codes developed in the North American region.

Long-span bridge loading

For loading of long span bridges, the uniformly distributed loading component has the greatest impact to the design effects. The AS5100, Eurocode and the UK national annexes have the highest uniformly distributed loading component of the codes and this is demonstrated by comparing the bending moments for spans of 50m up to 100m shown in figure A.31. The New Zealand loading is towards the middle of the range in terms of magnitude of the load.

A comparison of the maximum bending moments for simply supported spans at the ULS and including lane modification factors for three lanes of loading is shown in figure A.32. Excluding the Australian standard AS5100, the bending moments for the New Zealand loading at the ULS appear to fall comfortably within the other international design codes. This confirms the conclusions made in the Opus (2001 and 2011) reports which state that the current loading at the ULS is adequate. The Australian standard clearly shows a departure from the other loading codes which reflect the approach taken to develop a vehicle loading standard that represents an upper limit to account for future bridge loading rather than calibrating the design loading based on WiM data collected from current bridge loading.

Figure A.32   Comparison of ULS sagging moment for three lanes loaded (DLA inc)
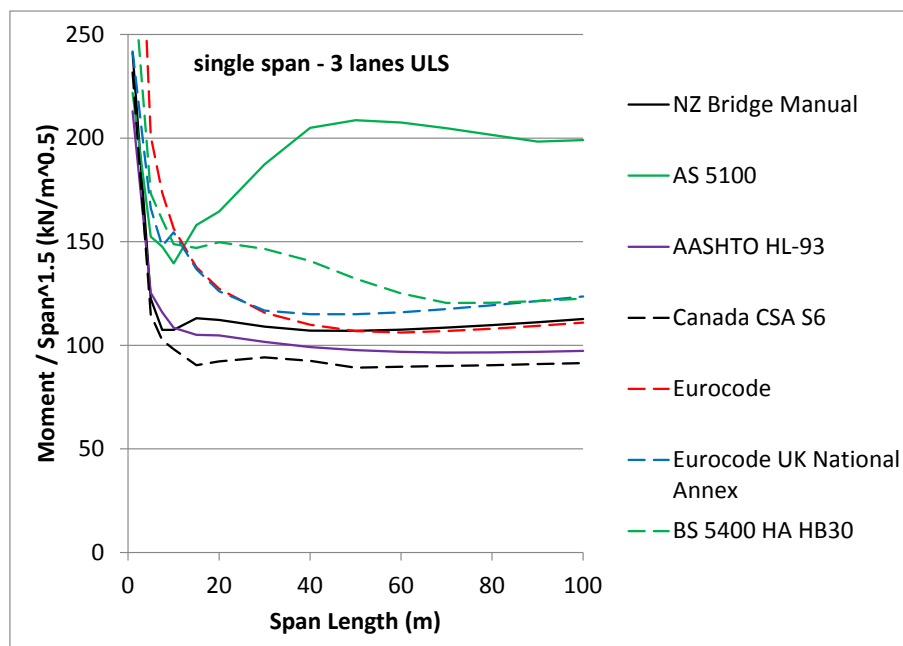
Continuous bridges

For continuous spans, a critical load case occurs where adjacent spans are loaded with traffic, which induces negative moments and higher reactions at internal piers. The loading models for the international codes accommodate this scenario by the following:

• The AS5100 uses a variable spacing between the central tri-axle set, essentially splitting the design vehicle into two vehicles which are placed at the designer's discretion to achieve the worst design effects.

• The AASHTO code introduces an additional vehicle and reduces the loading to 90%. The additional vehicle is placed at the designer's discretion to achieve the worst design effects.

• The CAN/CSA-S6-06 does not include a special provision for an alternative vehicle arrangement and the uniformly distributed loading component is intended to cover the loading of adjacent spans to induce the design effects for continuous bridges.

• The Eurocode does not include a special provision for an alternative vehicle arrangement and the uniformly distributed loading component is intended to cover the loading of adjacent spans or the special vehicle in load model 3 may be specified to consider these effects.

• The UK *National annex to BS EN 1991-2* is similar to the Eurocode, except for load model 3. Where load model 3 is specified, the series of special vehicles in load model 3 includes a variable axle spacing between a group of axles of the vehicle to achieve the worst design actions for continuous bridges.

A10.3 Lane widths and number of lanes

In reviewing the lane widths and number of lanes used for the traffic loading of bridges of the international design codes, it is observed that each region has a different methodology to determine these parameters. There is a common trend of the design codes where the following occurs:

Design lane width ≤ step increment width for an additional lane ≤ standard traffic lane width

This is a reasonable approach as it is a common practice for bridges, as traffic volumes increase, to increase the number of lanes on the bridge by reducing the traffic lane and/or shoulder widths on the bridge.

The *Bridge manual* specifies a design lane width of 3m and is comparable to the other design codes. The standard traffic lane width is 3.5m and is consistent with the Austroads road design guide used in New Zealand for road geometry design. The step increment width for an additional lane, however, is 3.7m which is greater than the standard traffic lane width and is inconsistent with the approach of the other design codes.

A10.4 Lane modification factors

In reviewing the lane modification factors of the international design codes for multiple loaded lanes, two common trends are noticed.

The Australian code, Eurocode and the UK national annexes reduce the loading of the additional loaded lanes by applying a modification factor to the individual lanes. This results in some lanes being more heavily loaded than others. It is noted that the lane modification factors are already built into the loading in the Eurocode and the UK national annexes. The UK national annexes, however, differ from the Eurocode as the latter reduces the tandem system loading in each additional lane. The AASHTO code and the CAN/CSA-S6-06 reduce the loading for multiple loaded lanes by applying a load modification factor to all loaded lanes concurrently, effectively averaging the traffic loading across all lanes. The *Bridge manual* is consistent with this procedure.

The trend of applying lane modification factors to the additional individual lanes is consistent with the probabilistic design philosophy of load factors and load combinations where the critical loading is combined with an additional transient effect, which is reduced to account for the reduced probability of concurrent loading. However, other influences may contribute to the procedure used which may include road rules within the regions where heavy vehicles are located in particular lanes such as slow lanes or for passing through toll booths.

A10.5 Dynamic load factors

The DLAs are considered differently for each design code and are described by the following:

- For the *Bridge manual*, the DLA is dependent on the span length and ranges from a 30% increase to a 10% increase, for spans between 0 – 100m, where the higher factors occur for the shorter spans.

- The AS5100 follows the same approach to that used in the CAN/CSA-S6-06 where the DLA depends on the type of vehicle or number of axles used in the loading where no dynamic amplification is applied for stationary vehicles and increases to a 40% increase for axle loading which is typical for loading of short-span bridges.

- The AASHTO code applies a DLA to the vehicular component of the loading only and excludes any dynamic amplification for the lane loading. The DLA applies a constant increase of 33%; however, as the amplification only applies to the vehicular component, the effective dynamic amplification is equal to or less than 33%.

- The CAN/CSA-S6-06 applies a DLA depending on the vehicle loading or the number of axles used in the loading. The dynamic amplification increases the load from 25% to 40% where the higher dynamic effects apply to the lower number of axles used in the loading and is typical for shorter-span bridges. Similar to the AASHTO code, the dynamic amplification does not apply to the lane loading.

- The Eurocode includes the dynamic amplification within the loading and thus is independent of the span length or type of loading. For load model 3 however, a DLA is applied and this is dependent on the span length. The magnitude ranges from a negligible increase for long spans to a 40% increase for shorter spans;

- The UK *National annex to BS EN 1991-2* is similar to the Eurocode except for load model 3 where the DLA is dependent on the magnitude of the axle loading and ranges from a 7% increase for the heavier axle loads to a 20% increase for the lighter axle loads.

Studies on the dynamic effects (Austroads 2003) have demonstrated that the dynamic increase is dependent on a number of factors. These include but are not limited to the:

- type of vehicle, mass and speed

- road profile

- span length

- frequency and damping of the bridge

- number of axles loaded on the bridge

- number of lanes loaded on the bridge.

In reviewing the codes there is a common trend for a maximum dynamic increase of 40% and this is typical for either shorter-span lengths or the type of loading for shorter-span lengths. There is also a common trend for the dynamic increase to reduce towards zero for longer-span bridges. This is achieved through

the design codes by two ways. One is through an equation, where the factor reduces with increased length. The other is where the uniformly distributed loading component, which has no dynamic factor applied, becomes the dominant component for longer-span bridges. For medium-span bridges, the DLA is typically in the range of 25% to 35%. The main trend of the codes, however, is to apply a simplified procedure for determining and applying the DLAs.

A10.6 Horizontal loads

Braking forces
The braking force is typically determined as a proportion of the weight of the vehicle or total loading. Table A.37 presents a generalisation between the codes of how the braking forces are determined.

Table A.37   Summary of braking forces in design codes

Design code	% of vehicle	% of total live load
Bridge manual	70%	10
AS5100	45%	15
AASHTO	25%	5
CAN/CSA-S6-06	28%	10
Eurocode	60%	10
UK *National annex to BS EN 1991-2* – load model 3	50%	N/A

It is observed that the North American codes derive the lowest braking forces for the associated design vehicle.

If the comparison is made between the New Zealand code, Australian code, Eurocode and the UK *National annex to BS EN 1991-2* for the ULS braking force as a percentage of the vehicle, the results are shown in table A.38.

Table A.38   Summary of braking forces in design codes with ULS factor included

Design code	% of vehicle	ULS load factor	% of unfactored live load
Bridge manual	70%	1.2	84%
AS5100	45%	1.8	81%
Eurocode	60%	1.35	81%
UK *National annex to BS EN 1991-2* – load model 3	50%	1.35	68%

The Australian code determines the braking force based on the capacity of braking systems and the friction between tyres and the pavement. The minimum requirement for braking systems is to achieve a minimum deceleration of 0.45g with evidence of vehicle tests showing braking systems achieving decelerations of 0.75g. Based on this procedure, the above design codes are reasonable for determining the braking force for a single vehicle for the SLS and ULS. The trend of the codes also includes a provision for larger volumes of traffic braking on bridges, which is more typical for longer bridge lengths, where the braking force is taken as 5% to 15% of the total live load.

Centrifugal forces
All codes have a common approach in basing the centrifugal force as a proportion of the live load where the proportion is based on the following physics equation for centripetal acceleration, a:

$$a = \frac{v^2}{r}$$

where: v is the speed

  r is the radius

A10.7 Load factors

Serviceability limit state

In reviewing the international design codes, the magnitude of the live load factor for the SLS for the normal traffic load combination follows a trend in using a factor close to or equal to 1.0. For other load combinations where the traffic loading is included but is not the critical transient load, the approach is to reduce the load factor to below 1.0 to represent average traffic effects.

Table A.39 below compares the SLS load factors for the normal traffic loading. Table A.40 compares the load factors, or an example of a load factor, where the traffic loading is an additional transient load.

Table A.39 Summary of SLS load factors in design codes

Design code	SLS load factor (normal traffic loading)
Bridge manual	1.35
AS5100	1.0
AASHTO	1.0
CAN/CSA-S6-06	0.9
Eurocode	1.0
UK *National annex to BS EN 1990*	1.0

Table A.40 Summary of SLS load factors for additional transient load in design codes

Design code	SLS load factor (traffic – additional transient load)
Bridge manual	1.35
AS5100	0.7
AASHTO	0.8
CAN/CSA-S6-06	0.9
Eurocode	0.75
UK *National annex to BS EN 1990*	0.75

It is clear that the load factor used by the *Bridge manual* departs from the common trend of the other design codes where the load factor is equal to or close to unity. It should be noted however that this was a result of previous studies where the SLS traffic loading model was assessed to be too low for actual traffic loading based on analysis of WiM data. Consequently the load factor (which was originally 1.0) was increased in 2004 to the value of 1.35.

It is proposed, at the conclusion of this research project, that a traffic loading model will be developed which includes a serviceability live load factor of 1.0 for the design of new bridges, in order to be consistent with trends of the other international design codes and so that the design load is consistent with the typical loads the bridge is designed to carry. This will subsequently lead to an increase in the unfactored loading from the current design load.

Ultimate limit state

In reviewing the international design codes, the magnitude of the live load factor applied to the nominal load at the ULS for the normal traffic load combination varies from 1.35 to 2.25. The North American and Australian codes show a trend to apply a load factor of 1.7 to 1.8. The European codes use a lower factor of 1.35. The load factor in the UK *National annex to BS EN 1990,* annex A2of 1.35 is lower than that specified in BD 37/01 which the loading was calibrated against; however, as shown in figures A.31 and A.32, the resulting ULS bending moments for the UK national annexes are still greater than those from BD 37/01 (shown as BS 5400 in the figures).

For other load combinations where the traffic loading is included but is not the critical transient load, the trend is to reduce the ULS load factor.

Table A9.5 below compares the ULS load factors for the normal traffic loading and the ratio of the ULS and SLS loading. Table A.41 compares the load factors, or an example of a load factor, where the traffic loading is an additional transient load.

Table A.41   Summary of ultimate limit state load factors in design codes

Design code	ULS load factor (normal traffic loading)	ULS Factor / SLS Factor
Bridge manual	2.25	1.67
AS5100	1.8	1.8
AASHTO	1.75	1.75
CAN/CSA-S6-06	1.7	1.9
Eurocode	1.35	1.35
UK *National annex to BS EN 1990*	1.35	1.35

Table A.42   Summary of ULS load factors for additional transient load in design codes

Design code	ULS load factor (traffic – additional transient load)
Bridge manual	1.35
AS5100	1.0
AASHTO	1.35
CAN/CSA-S6	1.4
Eurocode	1.01
UK *National annex to BS EN 1990*	1.01

The calibration of the ULS load factors for traffic loads depends on a number of factors. With developments of technology in transportation, the accuracy in measuring vehicle weights and axle loads has increased and there has been a historical trend for the codes to reduce the load factors for traffic loading as the probability of the traffic loading exceeding design limits may be calculated with greater precision. Studies carried out for the development of traffic loading in Australia (Pearson and Bayley 1997) demonstrated there is an upper limit of vehicle loading where the vehicles weight, or mass per length, is governed by volume limitations. Thus as traffic loading models in the design codes increase towards this upper limit (or volume limitation), there is a lower probability of the limit being exceeded.

Another factor which is taken into consideration is the magnitude of the dynamic amplification. Studies (Austroads 2003) carried out in Australia and New Zealand on the dynamic interaction of vehicles and bridges has demonstrated that the roughness of the road or irregularities, such as a cracked expansion

joint, can induce a dynamic amplification of 100% of the static load. Research carried out for the development of the UK *National annex to BS EN 1991-2* (Atkins 2005) describes that the derivation of BD37 includes an impact factor of 1.8 or dynamic amplification of 80%. The Australian code accommodates excessive dynamic amplification in the ULS load factor for traffic. In addition, it is also demonstrated in Austroads (2003) that as the mass of the vehicle increases, the dynamic amplification reduces. Other trends of the design codes are to calibrate the load factors based on a reliability analysis, or β factors.

A10.8 Evaluation

The evaluation of bridges in the international design codes follows a procedure to determine a rating factor where the factor is determined as a proportion of the live loading used in the evaluation. The general expression below is used to determine the rating factor for a particular element:

$$Rating\ Factor = \frac{Capacity - (Dead\ Loads + Other\ Permanent\ Effects)}{Live\ Load}$$

The capacity, dead loads and live loads all include the appropriate load factors or load reduction factors for the ULS or SLS.

The North American codes tend to use a three-level approach where the first level is to assess the bridge against the design load. Bridge performance can be assessed against the current standard and for the as-inspected condition of the bridge to determine whether the bridge can maintain operation without restriction. The second level is to assess the bridge against a lower loading than the design load, such as legal loads, to determine whether the bridge can maintain operation under certain load restrictions and the bridge is typically required to be posted with a load restriction. The third level is to specifically assess the bridge for a permit load where the actual permit vehicle is used in the assessment. This is carried out by road authorities for vehicles above the legally established weight limitations. In this case, additional restrictions may be placed on the vehicle in order to allow safe passage.

The evaluation procedure in the UK is based on the loading from the previous standard BS 5400 and is not updated to include the modified loadings as per the UK *National annex to BS EN 1991-2*. However, the loading in the UK *National annex to BS EN 1991-2* was calibrated to match the previous loading standard and is considered to be an adequate document until future replacement by a Eurocode standard. The assessment procedure differs from the other international standards where the capacity is simply compared with the design effects rather than calculating a rating factor.

A10.9 Conclusions

Our review indicates that the current New Zealand traffic loading standard for highway bridges is towards the middle of the range in terms of magnitude of the load when compared with other international design codes for all span lengths.

Based on the findings of this literature review and comparisons made between the *Bridge manual* and other international design codes, the researchers recommend that the following be considered in the development of the new vehicle loading standard for New Zealand:

• The traffic loading model should include a uniformly distributed load in conjunction with a design vehicle.

• The SLS load factor for the traffic loading model should be adopted as 1.0.

• To address the design of short spans, the traffic loading model should include a single axle loading to represent an individual overloaded axle.

- The wheel loading pressure should be consistent with the other design codes.

- The step increment for additional design lanes should be less than or equal to the standard traffic lane width of 3.5m.

- The DLA should be reassessed to be consistent with other international codes.

- The specification of the braking and centrifugal forces should be retained as a proportion of the weight of the traffic loading;

- Based on findings from this literature review, the live load factor at the ULS appears to be adequate for current bridge loadings. However, this is subject to economic evaluation and any allowances made for future increases to bridge loadings. It is recommended that the live load factor at the ULS should be chosen to achieve similar or greater design action effects as the current *Bridge manual*; however, it is recommended that the factor is chosen to achieve design actions which are not less than the current *Bridge manual*.

- The current calculation procedure for evaluation should be retained. The magnitude of the live load to be used in the evaluation should be reviewed once the new vehicle loading model has been developed.

A11 References

Agarwal, A, J Billing, P Buckland and R Pion (1993) *Proposed design truck models for the Canadian highway bridge design code.* Task Force on Live Loads, CHBDC Subcommittee on Loads and Load Factors.

Agarwal, A and M Cheung (1987) Development of loading-truck model and live-load factor for the Canadian Standards Association CSA-S6 code. *Canadian Journal Civil Engineering 14*: 58–67.

Agarwal, A and P Csagoly (1978) Evaluation and posting of bridges in Ontario. *TRR 664, Bridge engineering 1.* Washington DC: Transport Research Board.

American Association of State Highway and Transportation Officials (AASHTO) (2002) *Standard specifications for highway bridges.* 17th ed. Washington DC: AASHTO.

American Association of State Highway and Transportation Officials (AASHTO) (2011) *Manual for bridge evaluation.* 2nd ed. Washington DC: AASHTO.

American Association of State Highway and Transportation Officials (AASHTO) (2012) *AASHTO LRFD bridge design specifications.* 6th ed. Washington DC: AASHTO.

Atkins Highways and Transportation (2005) *Background to the UK national annexes to EN1990: basis of structural design – annex A2: application for bridges, EN1991-2: traffic on bridges.* Prepared on behalf of The Highways Agency.

Austroads (1992) *Bridge design code.* Sydney: Austroads.

Austroads (1997a) *Economic impacts of higher bridge design loadings, costs of providing stronger bridges.* Austroads Project RUM.H.96 working paper. Sydney: Austroads.

Austroads (1997b) *Future directions for Australian freight vehicles and bridge design traffic loading.* National Workshops summary information. Sydney: Austroads.

Austroads (2003) *Dynamic interaction of vehicles and bridges.* Austroads project no. T&E.B.N.508. Sydney: Austroads.

Austroads (2011) *Investigating the development of a bridge assessment tool for determining access for high productivity freight vehicles*. Austroads project no. FS 1580. Sydney: Austroads.

British Standards Institution (BSI) (2006) *BS 5400 part 2: Steel, concrete and composite bridges – part 2: specification for loads*. London: BSI.

British Standards Institution (BSI) (2008) *National annex to British standard BS EN 1991-2: 2003*. London: BSI.

British Standards Institution (BSI) (2009) *National annex to British standard BS EN 1990:2002+A1: 2005*. London: BSI.

British Standards Institution (BSI) (2011) *PD6688-2 Background to the national annex to BS EN 1991-2: Traffic loads on bridges*. London: BSI.

Bruls, A (1995) Evaluation of existing bridges under actual traffic. Pp829–834 in *IABSE reports 73*.

Bruls, A, J Calgaro and H Mathieu (1996a) ENV 1991 part 3: The main models of traffic loads on road bridges: background studies. Pp215–227 in *IABSE reports 74*.

Bruls, A, P Croce and L Sanpaolesi (1996b) ENV 1991 – part 3: Traffic loads on bridges: calibration of road load models for road bridges. Pp439–453 in *IABSE reports 74*.

Buckland, P (1981) Recommended design loads for bridges. *Journal Structural Division, ASCE*.

Bureau of Transport and Communications Economics (1997) *Roads 2020*. Roads 2020 working paper 35, Canberra.

Bushby, MB (2004) Bridge asset management in NSW, *Austroads 5th Bridge Conference*, 2004.

Calgaro, J and G Sedlacek (1992) EC1: Traffic loads on road bridges. Pp81–87 in *IABSE reports 65*.

Canadian Standards Association (CSA) (2006) *CAN/CSA-S6-06 Canadian highway bridge design code*. 10th ed with amendments 2010. Ontario: CSA.

Chapman, H, A Kennaird, J Huizing, P Stanford and J Wood (1974) Changes in New Zealand highway bridge design. *New Zealand Engineering 29*, no.6: 159.

European Committee for Standardisation (CEN) (2003) *EN 1991-2: Actions on structures – part 2: traffic loads on bridges*. Brussels: CEN.

European Committee for Standardisation (CEN) (2005) *EN 1990:2002+A1:2005: Basis of structural design*. Brussels: CEN.

Flint, A and B Jacob (1996) Extreme traffic loads on road bridges and target values of their effects for code calibration. Pp469–477 in *IABSE reports 74*.

Gordon, R and G Boully (1997) Revised bridge design live loads. *Austroads Bridge Conference*, Sydney, Australia.

Gordon, R and G Boully (1998) Bridges – planning for the future. *5th International Symposium on Heavy Vehicle Weights and Dimensions*, Australia.

Heywood, R (1995a) Live loads on Australian bridges – representative vehicles from weigh-in-motion data. *Australian Civil Engineering Transactions CE37,* no.2: 117–126.

Heywood, R (1995b) Live loads on Australian bridges – statistical models from weigh-in-motion data. *Australian Civil Engineering Transactions CE37*, no.2: 107–116.

Heywood, R, R Gordon and G Boully (2000) Australia's bridge design load model: planning for an efficient road transport industry. *Transport Research Record 1696, paper* 580034: 1–7.

Highways Agency (2001a) *Design manual for roads and bridges* (DMRB) *volume 1, section 3, part 14 BD 37/01 Loads for highway bridges.* London: Highways Agency, Department for Transport.

Highways Agency (2001b) *Design manual for roads and bridges* (DMRB) *volume 3, section 4, part 3 BD 21/01: The assessment of highway bridges and structures.* London: Highways Agency, Department for Transport.

Highways Agency (2011a) *Design manual for roads and bridges* (DMRB) *volume 3, section 4, part 19 BD 86/11. The assessment of highway bridges and structures for the effects of special types general order (STGO) and special order (SO) vehicles.* London: Highways Agency, Department for Transport.

Highways Agency (2011b) *Interim advice note 124/11: Use of Eurocodes for the design of highway structures.* London: Highways Agency, Department for Transport.

Hwang, E and A Nowak (1991) Simulation of dynamic load for bridges. *ASCE Journal of Structural Engineering 117*, no.5: 1413–1434.

Kirkcaldie, D and J Wood (2008) Review of Australian standard AS5100 bridge design with a view to adoption. *NZ Transport Agency research report 361.*

Kulicki, J, A Prucz, C Clancy, D Mertz and A Nowak (2007) Updating calibration report for AASHTO LRFD code. *Final report NCHRP 20-7/186.* Washington DC: Transportation Research Board.

Kwon, O and E Kim (2011) Calibration of live load factor in LRFD design guidelines. *Report OR 11-003.* Jefferson City: Missouri Department of Transportation.

Ministry of Transport (2010) *Regulatory impact statement – Land Transport Rule: Vehicle Dimensions and Mass Limits 2002 Amendment.* Wellington: Ministry of Transport.

Ministry of Works and Development (MOW) 1972 *Highway bridge design brief.* Wellington: MOW.

Moses, F (2001) Calibration of load factors for LRFR bridge evaluation. *CHRP report 454.* Washington DC: Transportation Research Board.

National Road Transport Commission (NRTC) (1996) *Mass limits review – report and recommendations of the steering committee.* Melbourne, Australia: NRTC.

New Zealand Transport Agency (2005) *Overweight permit manual.* 1st edition 2005. Wellington: NZ Transport Agency.

New Zealand Transport Agency (2010) *Economic evaluation manual.* Volumes 1 and 2. Wellington: NZ Transport Agency.

New Zealand Transport Agency (2013) *Bridge manual.* 3rd ed. Wellington: NZ Transport Agency.

Nowak, A and Y Hong (1991) Bridge live load models. *ASCE Journal of Structural Engineering 117*, no.9: 2757–2767.

Nowak, A (1993) Live load model for highway bridges. *Structural Safety 13*: 53–66.

Nowak, A (1999) Calibration of LRFD bridge design code. *NCHRP report 368.* Washington: Transportation Research Board.

O'Brien, E, B Enright and C Caprani (2008) Implications of future heavier trucks for Europe's bridges. *Transport Research Arena Europe 2008*, Ljubljana.

Opus International Consultants (2001) *Bridge manual load factors update.* Final report. Wellington.

Opus International Consultants (2011) *Vehicle Dimensions and Mass Rule change: development of bridge design loading for HPMVs*. Wellington.

Pearson, R and C Bayley (1997) Economic impacts of higher bridge design loadings, stage 1: vehicle alternative and benefits. *Austroads project RUM.H.96 working paper*.

Pearson, R (2007) *Review of the potential for increasing transport productivity through concessions on heavy vehicle mass and dimension characteristics*. Report to Ministry of Transport and Transit New Zealand.

Prat, M (2002) Traffic load models for bridge design: recent developments and research. *Prog. Structural Engineering Materials 3*: 326–334.

Sleath, L (1995) Investigation into the feasibility of heavy transport routes in New Zealand. *4th International Symposium on HV Weights and Dimensions*, Ann Arbor, USA.

Standards Australia (SA) (2004) *AS5100 Bridge design.* Sydney.

Standards Australia (SA) (2007) *AS5100.2 supplement 1, Bridge design – design loads – commentary.* Sydney.

Stimpson and Co (2011) *Monitoring, evaluation and review of the Vehicle Dimensions and Mass Rule, May 2010 – April 2011, part A – summary report.*

Taramoeroa, N and J de Pont (2007) Optimisation of heavy vehicle performance. *NZ Transport Agency research report 387*.

Travers Morgan (1995) Heavy transport routes: their identification and evaluation of a pilot route. *Transfund NZ research report 044*:

Wanty D and L Sleath (1998) Further investigations into the feasibility of heavy transport routes in New Zealand. *5th International Symposium on HV Weights and Dimensions*, Maroochydore, Australia.

WS Atkins Consultants (2002) Development of a loading model for abnormal vehicle effects. *BD86 background report no. CJ0054/BD86.* Epsom: Highways Agency.

Appendix B: Economic analysis

B1   Introduction

There is a significant variation in gross vehicle weight limits between countries for equivalent classes of heavy vehicles.

Heavy vehicle limits at any point in time reflect judgements about:

* the strength of existing bridges and pavements

* heavy vehicle performance and safety.

New Zealand has an extensive road network for its size and population. For this to be affordable, many road pavements and bridges were historically built to lower design loads than in more densely trafficked countries and heavy vehicle axle and gross weight limits have been correspondingly lower to minimise wear on pavements and bridges.

Mass limits are increased periodically, usually informed by studies into:

* increasing truck sizes and potential productivity benefits to transport operators

* the ability of existing bridges to handle heavier vehicles

* incremental bridge and pavement maintenance and strengthening costs

* heavy vehicle performance and safety.

Studies of proposed increases in heavy vehicle mass limits in Australia and New Zealand have consistently found that the benefits of allowing heavier trucks outweigh the costs, and hence limits should be increased. However, the studies have almost always only considered modest increases in vehicle mass limits, due to existing infrastructure constraints.

Because bridges have long lives, it is particularly important that bridges built today are designed to match the heavy vehicle mass limits that might potentially be economically viable in the long term.

B2   Austroads research

In 1996 Austroads commissioned a project to investigate the economics of higher bridge design loadings. The project aimed to look well beyond previous incremental increases in legal loading to try to find an end point in the evolution of the Australian general freight vehicle. The project found that optimal gross and axle mass limits were almost double existing mass limits. In other words, it was worthwhile increasing bridge design loadings significantly to ensure that bridges built today will have the strength to carry all potential future vehicles. The benefits are very large – even in 1994/95 dollars and with the lower freight volumes being transported at that time, benefits were up to $1 billion per annum – far greater than any incremental bridge and pavement costs.

Austroads noted that the only immediate effect of an increase in bridge design load is to increase the cost of constructing bridges. Given that potential benefits are speculative and well into the future, there will naturally be a reluctance to invest excessive amounts in the immediate future to provide this.

The additional bridge construction expenditure in the short term is analogous to purchasing a series of options to be able to realise a potentially large benefit in the future. How much it is worth spending on these options depends on the value and timing of the future transport efficiency benefits. An alternative

way to obtain similar benefits could be to strengthen all existing bridges at such future time as it is decided to increase heavy vehicle mass limits but that is not the subject of this study.

B3   Potential future vehicles

The Austroads project considered potential future vehicle configurations and mass and adopted the following two classes of vehicles to represent the various alternatives:

- vehicles at present maximum length – L group vehicles

- shorter vehicles with better swept path performance – S group vehicles.

The L group vehicles were constrained to remain broadly within current maximum length limits. Existing dimension limits, including height and width, were seen as so intrinsic to the total road infrastructure that they were unlikely to significantly change.

Table B.1 shows the lengths of different vehicle types in the two vehicle length classes.

Table B.1   Vehicle lengths

Vehicle type	L group length (m)	S group length (m)	Criteria for determining S group lengths
Articulated vehicle	19.0	14.8	Same swept path as 12.2m rigid truck
B-double	25.0	21.4	Same swept path as 19m articulated vehicle
B-triple	33.0	27.9	Same swept path as 25m B-double

The study calculated the freight density corresponding to the threshold between volume and mass constrained loads at existing mass and dimension limits to be 0.28t per cubic metre. Separately it determined that the maximum mass should be based on a freight density of 0.73t per cubic metre as fully loaded vehicles with freight densities of more than this were judged to have stability problems, and few freight types were denser than 0.73t per cubic metre in any case. For the analysis three intermediate load densities between 0.28 and 0.73t per cubic metre were also selected.

These load densities were applied to the different length vehicles and axle set and gross masses were calculated. The gross masses were much higher than the existing mass limits.

The study then calculated the benefits that could be achieved from future higher mass limits. This involved estimating the likely utilisation of higher mass limits and transfers between vehicle configurations. Benefits would arise because of the reduction in vehicle travel due to higher payloads. The savings from reduced travel of the fleet far outweighed the increased operating costs of the heavier vehicles.

Table B.2 shows the benefits for the L group (legal length) vehicles at higher payloads.

Table B.2 Benefits from higher payloads – L group (A$million pa, resource costs, 1994/95 dollars)

Group	Nominal payload on articulated vehicle (tonnes)	Gross benefit	Relative benefit (per tonne of increase in payload on articulated vehicle)
L28 (existing)	26.5	-	-
L43	40	587	43
L51	48	789	37
L58	54	875	31
L73	70	1,099	25

Table B.2 illustrates the following points:

- The benefits are very large – even in 1994/95 dollars and with the lower freight volumes being transported at that time benefits were up to A$1 billion per annum.

- There are diminishing returns to further increases in payload at higher loads. This is due to the nature of the Australian freight task. As the allowable mass gets larger, the percentage of the freight that is of sufficient density to allow vehicles to utilise this additional allowance continually diminishes.

- Further analysis indicated that benefits flatten off above 40t–45t payload (on an articulated vehicle) and increased payload above 70t yields little incremental benefit.

Similar results and trends were found for the S group vehicle configurations.

The Austroads study also considered vehicle stability, safety and driveability of the potential higher mass vehicles, and the cost of increased pavement wear.

Both current and potential technological changes in vehicle design were assessed for a range of operational criteria including lateral stability, braking, ability to maintain speed on grades and high-speed dynamic off tracking. This work indicated that acceptable vehicle stability and driveability could be readily achieved at the L43 level, could be achieved with more effort at the L51 level, and could possibly just be achieved with considerable inclusion of new technology at the L58 level. It would be unlikely that L73 vehicles would be viable.

The pavement assessment found that pavement wear would not be expected to be a compelling reason to prevent increased axle set mass until at least the 30t–35t (L43–L51) level but would be expected to rule out increases beyond the 40t (L58) level.

Based on the above assessments it was concluded that a suitable envelope upon which to base the bridge design loading model would be the L58 long and S73 short vehicle groups as detailed in table B.3.

Table B.3   Maximum likely feasible future vehicle configurations and mass

Group title	Vehicle	Length (m)	GVM (tonnes)	Axle set loadings		
				Steer	Dual	Tri-axle
L58 (freight density 0.58t/cu m)	Articulated	19.0	74	7	27	40
	B-double	25.0	114	7	27	40
	B-triple	33.0	154	7	27	40
S73 (freight density 0.73t/cu m)	Articulated	14.8	73	7	26.5	39.5
	B-double	21.4	112.5	7	26.5	39.5
	B-triple	27.9	152	7	26.5	39.5

The above summary describes how the traffic loading envelope was developed for the current bridge design load in Australia based on long-term economic considerations

B4   Bridge construction costs

As part of the Austroads project, bridge engineers from three Australian state road agencies collaborated to estimate construction costs for bridges designed for a range of higher design loads and compare these with average costs for existing bridge designs to determine the incremental capital costs of higher design loads.

Construction cost increases were assessed relative to both current main traffic design loadings at the time:

- The T44/L44 design loading – the basis of allowing for general access vehicles. This aimed to simulate a single heavy articulated vehicle, with longer vehicles and multiple heavy vehicles within the traffic stream being simulated by the L44 lane loading.

- The HLP400 loading – a large proportion of new bridges were being designed to provide HLP320 or HLP400 capacity in some, if not all, lanes. This is a more severe loading than T44/L44 for most spans.

Construction cost increases were assessed by estimating differential costs of providing for a range of vehicle masses and configurations when compared with both of the above current (at that time) design traffic loads. This comparison was carried out for a range of bridge spans and construction forms, ranging from 3m crown units to 45m box girders to reflect the major areas of Australian bridge construction. Summary results of this are shown in figure B.1. To allow for the effect of the two alternative traffic design loadings used, the cost increases have been calculated for a weighted average of the two. Axle set loadings are used to allow each group of vehicles to be plotted as a single point.

Figure B.1   Estimated increases in bridge construction costs
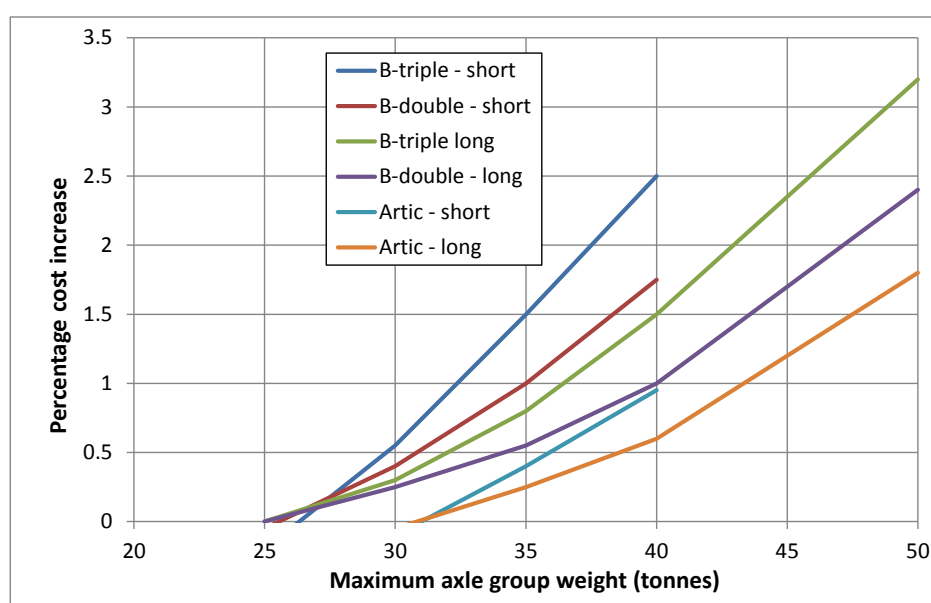
Note: Lines on graph are in same order as legend

Figure B.1 demonstrates several features:

- Bridge construction costs are not highly sensitive to increases in design vehicle mass, increasing approximately 1% for each 10t increase in axle set loading.

- Bridge construction costs also vary with the configuration of the vehicle, thus moving from a semi-trailer (artic) as the design vehicle, to a B-double, and then to a B-triple, results in further increases of around 0.5% at each step.

- Bridge construction costs also vary slightly with the length of the design vehicle.

The study concluded that a minimal increase in bridge expenditure of 2%–3%, representing an annual cost increase of approximately A$4 million to A$7 million across Australia, would provide for a very substantial increase in bridge design load.

Back-calculating from these estimates suggests that bridge construction expenditure in Australia at the time (1997) totalled approximately $220 million per annum. This is consistent with the amount reported during Australia's heavy vehicle charges determination at the time.

B5   Application of Austroads project results to New Zealand

It should not be assumed that the Austroads project findings are directly applicable to New Zealand. The vehicles in table B.3 are likely to represent an **upper bound** of the potential future vehicle loadings that should be adopted in determining the new bridge design load standard in New Zealand.

Benefits from New Zealand studies into increased mass limits (see appendix A) have tended to be considerably less than obtained in the Australian studies. Possible reasons for this include a smaller freight task and fewer trucks in New Zealand and a smaller proposed increase in vehicle loads in the studies reviewed.

New Zealand's road user charges (RUC) system is also likely to have an influence on future heavy vehicles. RUC rates increase exponentially for heavier vehicles. This is likely to diminish the attractiveness to transport operators of moving to the much heavier trucks and axle loads operated in other countries, unless the basis of determining RUC rates changes to a more linear scale.

These and other factors mean that the optimal upper limit of vehicle loads is likely to be lower for New Zealand than most other developed countries, at least for as-of-right higher limits on all roads. However if the incremental cost of building stronger bridges is small it may be reasonable to err on the high side rather than the low side if there is uncertainty about the most economic design load.

The original Vehicle Dimensions and Mass Rule Amendment proposal included the option of an across-the-board increase in gross mass to 50t from the current 44t gross mass limit and an increase in permitted vehicle length to 22m from 20m. This option was removed from the final rule amendment but it should be noted that both the Transit NZ heavy vehicle limits project and studies for the Vehicle Dimensions and Mass Rule Amendment showed it to be economically justified. It should therefore be regarded as the **lower bound** of potential future vehicles for a new bridge design load.

B6   Indicative economic assessment

An indicative economic assessment has been carried out based on the findings of previous studies and a number of assumptions about New Zealand bridge designs and current bridge construction expenditure.

New Zealand bridge construction and renewal expenditure is assumed to average NZ$150 million per annum (in current prices) over the next 30 years based on existing bridge design loads and standards. Construction and renewal programs are shown on the NZ TA website but not in a way that makes it possible to determine the actual current expenditure on bridges. However, from the information available on the website, and from comparison with Australian expenditure, this appears likely to be a conservative (ie high) estimate.

Analysis is carried out for two bridge design loading scenarios as follows:

* 1% higher bridge construction costs (provides for 10t increase in maximum axle set weight)

* 2% higher bridge construction costs (provides for at least a 15t increase in maximum axle set weight).

These scenarios are derived from figure B.1 based on the following assumptions:

* Existing bridge design loadings and construction types in New Zealand are similar to those that existed in Australia in the mid-1990s or alternatively at least the strength and cost relationships are similar to those that were derived for Australia.

- The slopes of the curves in figure B.1 are applicable to New Zealand bridges. In other words the slope of the relationships between increasing strength and increasing cost are similar.

As noted above the original Vehicle Dimensions and Mass Rule Amendment proposal included an option of an across-the-board increase in gross mass to 50t from the current 44t gross mass limit. This represents an increase of less than 5t per axle set so is well within the first scenario above (ie 1% higher bridge construction costs).

The assessment assumes that the increase in bridge construction expenditure due to adopting a higher bridge design load is incurred every year for the next 30 years and that mass limits can only be increased and transport benefits realised after 30 years when a sufficient proportion of existing bridge stock has been replaced at the new standard. Transport productivity benefits are estimated for the following 30 years, ie until 60 years from today. This is considered conservative because in reality effort is likely to be concentrated on key routes and some benefits may begin earlier than 30 years from now. Successive 30-year periods are adopted as this is the Transport Agency's usual evaluation period for project appraisals.

Indicative assessment benefits are assumed to total NZ$100 million per annum, as shown in table B.2.

The Austroads project estimated annual benefits of A$875 million per annum (approximately NZ$1 billion) (in 1994/95 dollars) for the maximum feasible long configuration vehicle. This is equivalent to approximately NZ$1.5 billion in current prices. NZ$100 million is 6.7% of this, which seems reasonable for New Zealand. As reported in appendix A, the first phase of Transit NZ's heavy vehicle limits project estimated a NZ$450 million present value of total benefits (in 1996 dollars) over 30 years for an increased heavy vehicle load limit based on 1.15HN. This is equivalent to approximately NZ$60 million per year in current prices. Based on this, a benefit of NZ$100 million per annum for the highest likely long-term heavy vehicle weights appears reasonable.

The Transport Agency's discount rate for project appraisals is 8% (real) per annum. The NZ Transport Agency (2010) *Economic evaluation manual* indicates that lower discount rates of 4% and 6% can be used for evaluations of activities that have long-term future benefits that cannot be adequately captured with the standard discount rate. This would appear applicable to long-life assets such as bridges. The assessment has therefore been carried out at each of the suggested discount rates.

Table B.4 shows indicative results and benefit–cost ratios based on the above scenarios and assumptions.

Table B.4 Indicative economic assessment results (all costs in $ million)

Scenario	1% higher costs			2% higher costs		
Discount rate	8%	6%	4%	8%	6%	4%
Total bridge cost pa ($M)	150	150	150	150	150	150
Incremental bridge cost pa ($M)	1.5	1.5	1.5	3.0	3.0	3.0
Uniform series discount factor – over 30 years	11.26	13.76	17.29	11.26	13.76	17.29
Present value of incremental cost over 30 years	16.9	20.6	25.9	33.8	41.3	51.9
Future benefit per annum ($M)	100	100	100	100	100	100
Uniform series discount factor – over 30 years	11.26	13.76	17.29	11.26	13.76	17.29
Present value of benefits over 30 years ($M)	1126	1377	1729	1126	1377	1729
Discount factor for single amount at 30 years	0.099	0.174	0.308	0.099	0.174	0.308
PV of benefits over 30 years starting in year 30	111.9	239.7	533.1	111.9	239.7	533.1
Indicative benefit/cost ratio	6.6	11.6	20.6	3.3	5.8	10.3

Based on the assumptions discussed above, the indicative economic assessment shows that a significant increase in bridge design load is likely to be economically justified. An increase in bridge design load that accommodates an increase in maximum axle set weight of up to 10t is only expected to increase bridge construction costs by approximately 1% and this has a benefit–cost ratio of 6.6 – 20. Even if bridge construction costs are 2% higher and the benefits are half the NZ$100 million estimated, the benefits will still exceed the costs (BCR 3.3 x ½ = 1.6).

B7   Conclusion

Because bridges have long lives, it is particularly important that bridges built today are designed to match the heavy vehicle mass limits that might potentially be economically viable in the long term.

Research by Austroads in 1996 for Australia's bridge design load review found that economically optimal gross and axle mass limits were almost double existing mass limits. This study found that it was worthwhile increasing bridge design loadings significantly to ensure that bridges built today would have the strength to carry all potential future vehicles. The benefits are very large and are far greater than any incremental bridge and pavement costs.

Austroads found that bridge construction costs are not highly sensitive to increases in design vehicle mass, increasing approximately 1% for each 10t increase in axle set loading. Bridge construction costs also vary with the configuration and length of the vehicle, thus moving from a semi-trailer as the design vehicle, to a B-double, and then to a B-triple, results in further increases of around 0.5% at each step.

Austroads concluded that a minimal increase in bridge expenditure of 2%–3%, representing an annual cost increase of approximately A$4 million to A$7 million across Australia, would provide for a very substantial increase in bridge design load.

An indicative economic assessment carried out by applying the Austroads study findings to New Zealand, and subject to a number of assumptions, shows that a substantial increase in bridge design load is likely to be economically justified. An increase in bridge design load that accommodates an increase in maximum axle set weight of up to 10t is only expected to increase bridge construction costs by approximately 1% and would result in a benefit–cost ratio of 6.6. Such an increase in design load is much higher than any mass limit increases that have been considered in previous studies.

Because significantly higher bridge design loads only require a slight increase in bridge construction costs it would be worth considering a substantial increase in New Zealand's bridge design load to future-proof new and replacement bridges for all potential future heavy vehicle configurations and loads over the next 30 years, not just the types of HPMVs currently being proposed under the VDAM Rule Amendment 2010.

B8   References

Austroads (1997) *Economic impacts of higher bridge design loadings, costs of providing stronger bridges.* Austroads Project RUM.H.96 working paper.

Pearson, RA (1997) *Economic impacts of higher bridge design loadings, stage 1: vehicle alternatives and benefits.* Prepared for Austroads.

Wanty, D and L Sleath (1998) Further investigations into the feasibility of heavy transport routes in New Zealand. *5th International Symposium on HV Weights and Dimensions*, Maroochydore, Australia

Appendix C: Weigh-in-motion analysis

C1   Introduction

A detailed international literature review was undertaken as an initial task on this project covering both the economic and the engineering aspects of this task (appendix A). Key aspects of the literature review were an assessment of the economic benefits from increases in vehicle loading, and the various approaches taken internationally to the development of new vehicle loading standards.

This literature review identified that there are a number of different model representations for the design traffic load in the international standards reviewed:

- The Eurocode uses a double axle tandem and a uniformly distributed load.

- The AASHTO code uses a design truck or double axle tandem in combination with a uniformly distributed load.

- The Australian code uses two load models each comprising four tri-axles in combination with a uniformly distributed load. The code also includes a separate heavy axle or wheel loading.

- The Canadian code CAN/CSA-S6-06 uses a design truck or double axle tandem in combination with a uniformly distributed load.

With the exception of Australia, the development of the traffic load model was based on a review of WiM data and the choice of a configuration of the load model to produce similar load effects to that of measured vehicles. Variability of the design load (and hence choice of load factor) was determined based on traffic survey data and historical trends in vehicle weights and truck numbers, with consideration of likely variance over the design life of the structure.

The procedure, while complex, is reasonably well established. From experience we know that for most bridges (up to about 30m span), a single vehicle is the critical event, while for long-span bridges the traffic stream is critical. In brief, the methodology involves utilising WiM data to construct artificial queues of stationary vehicles on bridges. Critical inputs to this analysis are the sizes and frequency of loads analysed from the WiM data, together with future trends in loads. Traffic volumes provide information on the time for queue formation, and hence the frequency of critical loading events occurring. From this, the loading event with a limiting probability of occurrence can be determined.

This same approach was followed in this project. The next sections of this appendix describe the analysis undertaken of available New Zealand WiM data and the statistical analysis of this data that followed.

The WiM analysis presented in this appendix is an input to the model development described in appendix D. This appendix describes the process for analysing the loading effects from the WiM data. There is no discussion or interpretation of the WiM data, as it relates to the development of the loading model, in this appendix.

C2   Analysis of weigh-in-motion data

C2.1   WiM site locations

The Transport Agency has five WiM stations in New Zealand collecting axle load data on a continuous basis for use nationally in traffic monitoring. The WiM stations (according to the *2010 Annual weigh-in-motion report* issued in April 2011 (NZ Transport Agency 2011)) are described in table C.1 and their approximate locations are shown in figure C.1.

Table C.1   WiM station descriptions

Region	SH	RS	Description
02-Auckland	1N	461	DRURY – telemetry site 48 – (WiM site 1205)
03-Waikato	1N	625	TOKOROA – telemetry site 51 – (WiM site 421)
04-BayofPlenty	2	171	TE PUKE – telemetry site 49 – (WiM site 24)
06-Hawke'sBay	5	259	ESKDALE* – telemetry site 101 – (WiM site 5721)
11-Canterbury	1S	284	WAIPARA – telemetry site 52 – (WiM site 518)

*Eskdale WiM site data collection was started in July 2010.

Figure C.1   Approximate WiM station locations

C2.2   Assessment of data

The Transport Agency provided the researchers with WiM data from the above sites on 2 December 2011 dating back to year 2000 (data for the Eskdale site was not available). Details of the traffic volumes and classification of vehicles for each of the four longer-term sites are summarised as follows:

Table C.2   AADT and vehicle classification – Drury WiM site

Class	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
COUNT	36470	36207	38300	39734	40349	40333	41631	40987	41259	41402
CAR	91%	89%	89%	90%	90%	90%	90%	87%	89%	89%
LCV	1%	1%	1%	1%	1%	1%	1%	1%	1%	1%
MCV	3%	3%	3%	3%	2%	3%	3%	5%	3%	3%
HCV1	1%	2%	2%	2%	2%	2%	2%	2%	2%	2%
HCV2	4%	5%	5%	5%	5%	4%	5%	5%	6%	6%

Table C.3   AADT and vehicle classification – Te Puke WiM site

Class	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
COUNT	16531	17193	17786	18499	18223	18196	19007	18625	19273	19143
CAR	90%	89%	90%	91%	90%	90%	91%	89%	90%	89%
LCV	5%	1%	1%	1%	1%	1%	1%	1%	1%	1%
MCV	1%	3%	3%	3%	3%	2%	2%	4%	2%	2%
HCV1	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%
HCV2	2%	5%	5%	4%	4%	4%	4%	5%	5%	5%

Table C.4   AADT and vehicle classification – Tokoroa WiM site

Class	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
COUNT	8418	8644	8511	8839	8614	8400	8643	8798	8504	8470
CAR	90%	80%	80%	86%	85%	83%	84%	83%	83%	83%
LCV	5%	1%	1%	1%	1%	1%	1%	1%	1%	1%
MCV	1%	4%	4%	3%	4%	4%	3%	4%	3%	3%
HCV1	1%	3%	3%	2%	2%	3%	2%	3%	3%	3%
HCV2	2%	13%	13%	8%	8%	9%	10%	10%	10%	11%

Table C.5   AADT and vehicle classification – Waipara WiM site

Class	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010
COUNT	6107	6412	6741	7034	7144	7156	7626	7297	7431	7479
CAR	87%	87%	87%	87%	86%	85%	85%	83%	84%	83%
LCV	2%	2%	2%	3%	3%	3%	3%	2%	3%	3%
MCV	3%	3%	3%	4%	3%	3%	3%	4%	3%	3%
HCV1	2%	2%	2%	2%	2%	2%	2%	2%	2%	2%
HCV2	6%	6%	6%	5%	6%	7%	7%	8%	8%	9%

The WiM analysis was undertaken on vehicle classes 'MCV', 'HCV1' and 'HCV2' which range from light rigid trucks to large articulated trucks. These types of trucks made up between 10% to 15% of the total vehicle mix depending on location, of which HCV2 made up of 50% of the trucks. The trend showed that the number of trucks is increasing at all sites, and in most cases, has a substantially higher growth rate than cars.

A review of the data was undertaken to check that the WiM data used for the analysis was 'fit for purpose'. General quality assurance included removing duplicate rows in the data and assessing the overall 'quality' in terms of average weight, maximum, minimum and standard deviation. When there were spikes in the average, or there were extended flat periods in the data, this could suggest an error in the calibration of the WiM sensor. This assessment was used to select periods of data suitable for analysis. For more information, see the discussion later in this section.

Tests were also conducted on each event (or vehicle where applicable) as read from the original WiM data. The analysis software automatically flagged events that violated any of the conditions listed below. The conditions were developed based on recommendations from Enright and O'Brien (2011). These events were then manually inspected and removed if necessary. In many cases, events were retained even if they violated some of these conditions – they were only removed if they were obviously erroneous. Most of these conditions were never

violated, probably due to the data having already been cleaned by the Transport Agency before being received by the researchers. The conditions that were frequently violated are discussed below.

- A negative gap or a small gap (less than 5m) between the rear of a leading truck and the front of the following truck. This violation was relatively common, most likely due to one or both of the reasons listed below. These records were nonetheless retained.

  - Split vehicles may be present in the data, meaning that one vehicle (eg of eight axles) is recorded as two vehicles (eg of four and four axles) with a very small gap between them. This is not a problem from an analysis perspective as we are only interested in loads and spacings, not the number of discrete vehicles.

  - Gaps are determined by combining the length and speed of the vehicles with their time stamps. Available data only has time stamps accurate to the nearest second, which is not of sufficiently high resolution to calculate vehicle gaps with a high level of accuracy, leading to some negative or very small calculated gaps. The calculated gaps were deemed accurate enough for analysis purposes.

- A slow (less than 40km/h) or fast (greater than 150km/h) reading. Speeds outside this range may not be realistic or could affect the accuracy of the WiM readings.

- A light (less than 3.49t, to classify it as a truck) or heavy (greater than 100t) reading.

- A reading with less than two axles.

- A reading with a wheelbase (distance between the front and rear axle) of greater than 30m.

- A reading with a negative overhang – a wheelbase longer than the vehicle's total recorded length.

- A reading with a single axle greater than 20t. This occurred several times, leading to many readings being removed upon manual inspection, although some readings that violated this condition were retained. For more information, see the discussion in section C3.

- A reading with an individual axle spacing greater than 20m.

This analysis of the WiM data suggests that the data before 2006 appeared to be significantly less reliable than the data from 2006 onwards. The graph in figure C.2 is a typical example (from the Te Puke WiM site) where there appears to be erroneous variations in the first six years of data. The data from 2006 onwards, however, appears to be much more reliable for all of the sites.

Figure C.2   Steer axle diagnostics from the Te Puke site
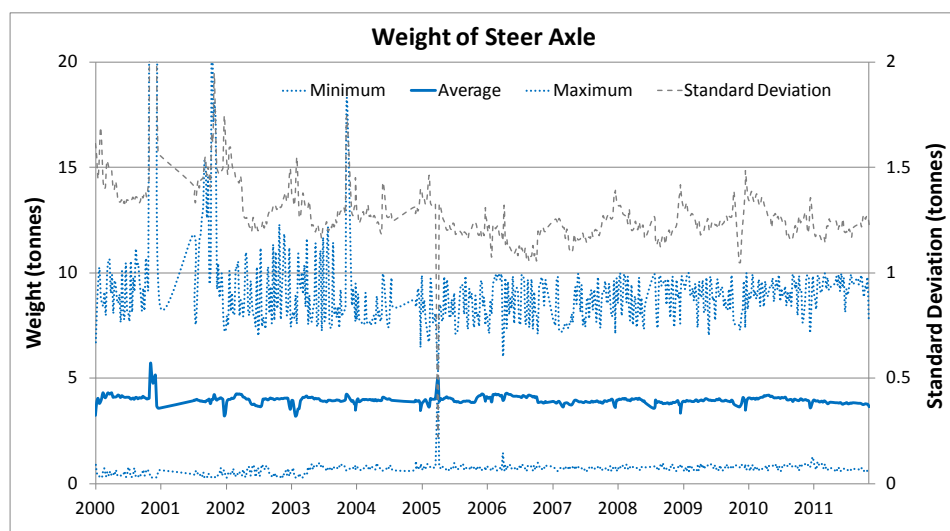
This apparent anomaly was queried with the Transport Agency, who advised that the accuracy of a WiM system is affected by a number of factors, principally the pavement condition. We understand that the pavement levels, structure and associated sensor installation have had a chequered history at each of the sites - the Te Puke, Tokoroa and Waipara sites underwent strengthening and other corrections around 2006 and the Drury southbound lane was resurfaced in 2004.

An analysis of a 'WiM rejects' file from the Transport Agency showed that many records had been removed prior to the researchers receiving the data. These records appear to be legitimately erroneous; however, the researchers only received rejected records from October 2010 to November 2011. The extent to which extreme values were removed from the data is unknown. However, subsequent analysis of the data showed that a significant number of days had been removed, or were not available, from the data, as shown in table C.6.

Table C.6   Number of 'missing data' days by site

	2006	2007	2008	2009	2010	2011*
Drury	0	0	14	0	123	14
Tokoroa	36	103	108	0	12	1
Te Puke	49	39	50	35	15	63
Eskdale	N/A	N/A	N/A	N/A	9	30
Waipara	0	65	2	0	1	1

* Note, data for 2011 was only available up to 22 November

C2.3   Load effects and loading events

Each lane of WiM data from the four WiM sites with 12 years of data (Drury, Tokoroa, Waipara and Te Puke) and from the one WiM site with only two years of data (Eskdale) was analysed separately. There were a total of 12 lanes between the five sites.

The data was grouped into a 'stream' of axles by site, by lane and by day. Each stream of axles was then divided into an 'event'. For the analysis using observed traffic flow conditions, an event was defined as a stream of axles separated by the length of a span, where the span length is a user input. For the queued traffic flow analysis, an event was defined as every record within a one-hour period, with each vehicle separated by a 3m gap between the rear axle of the leading vehicle and the front axle of the following vehicle. Separate events occur when the spacing of axles exceeds the span length.

Using this information the following load effects were calculated for a range of span lengths:

- maximum mid span moment for a single span (M_m)

- maximum shear at near support for a single span (R_A)

- maximum shear at far support for a single span (R_B)

- maximum moment at central support for a twin continuous span (M_B)

- maximum shear in near span at central support for a twin continuous span (V_{AB})

- maximum shear in far span at central support for a twin continuous span (V_{BC}).

Twin continuous span bridges have two spans of the specified length, ie a 10m twin continuous span bridge is 20m long in total. The span lengths were; 5m, 10m, 20m, 40m, 60m, 80m and 100m.

Figure C.3   Loading event

1. Axle stream 'slides' across the bridge span

2. Shears and moments are calculated for each event

3. An event is defined by a stream of axles passing over a span. A new event occurs after a gap of no axles is greater than the span length

4. The maximum shears and moments are calculated by site/lane/day
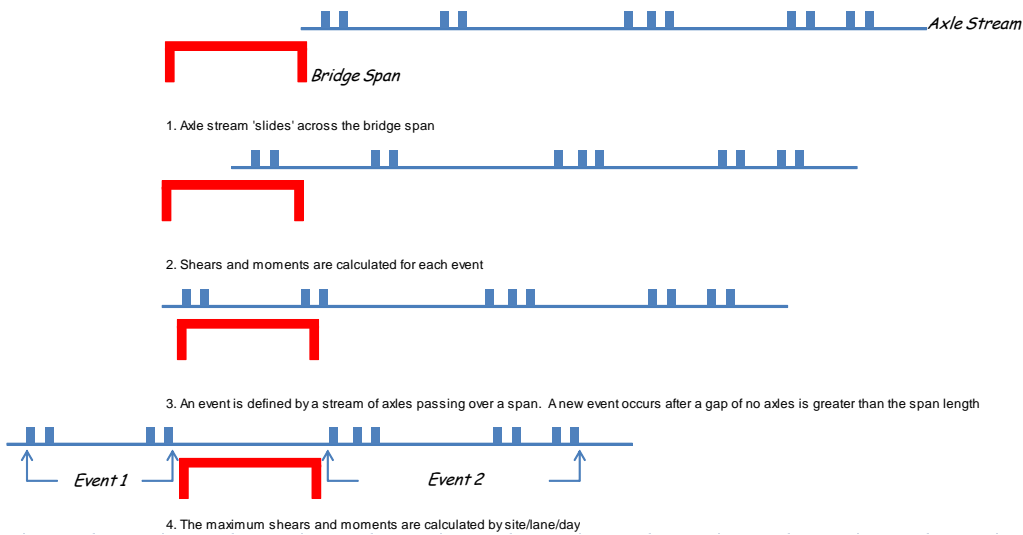
These maximum values for each day, lane and site were recorded in a simple output spreadsheet (six years is approximately 2190 rows of data per scenario) and further testing was carried out to check for:

- independence – each maximum was a specific random variable

- same distribution – a single distribution covers the period of analysis

- trend – that the maximums remain stable over time.

C2.4   Selection of analysis site

One site was selected to undertake the detailed analysis of the extreme values. As such the analysis site needed to be representative of all loading conditions. Selection of the analysis site was based on the average and maximum shear and moment values extracted from a preliminary loading analysis, which included all lanes at all sites for 2006 to 2011. Average values for each load effect were found to be very similar across all sites, however Drury 'lane 4' had higher maximum values. Being close to Auckland, Drury lane 4 also has a larger amount of traffic and particularly, heavy vehicles. Therefore, this site was chosen to conduct all statistical analysis of the extreme values.

Six years of data was available for statistical analysis from the preliminary analysis for the selected Drury lane 4 analysis. However, two distinct trends were present in the data prior to 2009, as shown in figure C.4, an example from the 10m span case. As the generalised extreme value (GEV) distributions require data with no trend, only the three most recent years of data (from 1 January 2009 to 22 November 2011) were used for the statistical analysis.

Figure C.4   Example trends from the 10m span case for moment at central support, twin span (M_B)
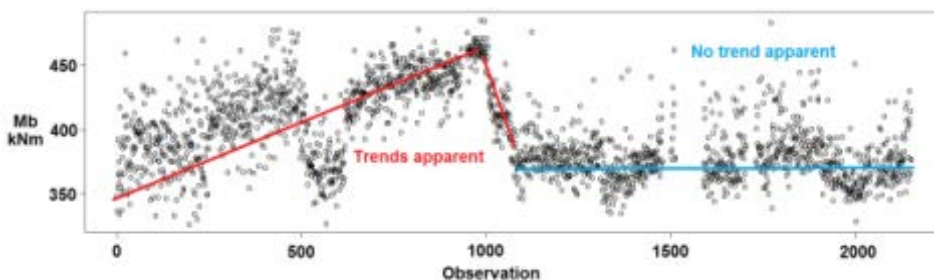
Nineteen days were identified as having very high maximum load effects. These were examined individually and it was deemed that 13 of them had load profiles that were physically impossible. All of these records had small axle loads with the exception of a single axle load that was greater than 20t. These were removed from the data. The other six heavy trucks were deemed to be realistic records with more even load profiles. One day (18 January 2010) only had one record, and was therefore also removed. The final dataset contained 905 data points.

C2.5   Data quality

As described in section C2.2, a review of the data was undertaken to check that the WiM data used for the analysis was 'fit for purpose'. Some anomalies were identified, and queried with the Transport Agency. The review resulted in data from the period 1 January 2009 to 22 November 2011 being used for the WiM analysis, with the remaining data from a six-year period being discarded.

Subsequent to the WiM analysis being completed, the Transport Agency provided the researchers with the Beca (2012) research report *Heavy vehicle data collection, analysis, and validation summary,* rev A. This report concluded that there appeared to be considerable variation in weight calibration accuracy over time and between lanes at all WiM sites, and that only 20% of the data was deemed to be usable. This conclusion means that the analysis of the WiM data, and the conclusions from that analysis, may be unreliable.

The information presented in this appendix is an input to the model development described in appendix D. Because of concern about the reliability of the WiM data the model development in appendix D does not rely on the WiM data to determine the recommended loading model. In fact the WiM data is only used in a minor way for the selection of an appropriate accompanying lane load. This is discussed further in appendix D.

C3   Statistical analysis

C3.1   Approach

A load effect on a bridge is any effect, such as bending moment or shear, which results from any form of loading event. The study of bridge loading effects is principally concerned with size and frequency of extreme or rare events. Or, in other words, the size and shape of the distribution tails.

Extreme value theory aims to characterise rare events and tails of distribution. The Fisher–Tippett theorem (Wikipedia 2013a) suggests that the block of maxima of a sequence of identically, independently distributed (iid) random variables belong to one of three distributions regardless of the original data:

1   Gumbel (light-tailed) distribution

2   Fréchet (heavy-tailed) distribution

3   Weibull (finite endpoint) distribution.

Collectively, this family of distributions is known as GEV distribution.

Many authors have approached this problem by identifying the maximum load effect observed during a 'block' of time and have shown that load effect data for the ith event type, Si, fits well with the GEV distribution (Caprani et al (2008), Nowak (1993) and Castillo (1988)).

The extRemes (ismev) package for the R statistical language was used to perform a GEV analysis (Wikipedia 2013b) on each load effect for each span length. The maximum likelihood estimate was determined for the three GEV distribution parameters; μ (location factor), σ (scale factor) and ξ (shape factor). Note that when $\xi = 0$, the distribution is a Gumbel distribution, and the return level plot on a logarithmic scale is a straight line. Other possible distributions are Fréchet ($\xi > 0$) and Weibull ($\xi < 0$)

distributions which produce lines that curve up and down respectively on the return level plot. Quantile plots were examined as well as p-values to determine whether the GEV distribution was a good fit for the data. P-values reflect the probability that the data points match the fitted distribution due to chance alone. If the p-value is less than 0.05, there is less than 5% probability that the fit is due to chance, and more than 95% probability that the data points actually come from the fitted distribution. Where the fit is poor ($p > 0.05$), it is noted.

Once the distribution parameters have been fitted to the observed data, the curves are extrapolated to predict the values corresponding to 5% probability of exceedance in one year (20-year return period) and 5% probability of exceedance in 100 years (2000 year return period).

This methodology has been undertaken separately for three scenarios, described below.

C3.1.1 Observed flow conditions scenario

The WiM records are run across the bridge spans as they actually happened, with the spacings between vehicles calculated from their speeds, lengths and record times. Light vehicles are ignored.

C3.1.2 4% queued flow scenario

This scenario is designed to test the effects of stationary queued conditions on potential bridge loads and is described below. The 5m and 10m bridge spans were not considered for the queued case as the majority of individual trucks are longer than 10m.

The number of trucks that cross a bridge under queue conditions is dependent upon factors including the average annual daily traffic (AADT), average annual daily truck traffic (AADTT), the ratio of AADTT to AADT, and congestion issues at the bridge location. Increases in AADT lead to greater probability of queued traffic; increases in AADTT lead to greater probability of queued trucks; and increases in the ratio of AADTT to AADT lead to fewer cars between the trucks (resulting in heavier queues). However, the greatest impact on the number of trucks that cross a bridge under queued conditions is congestion at the bridge location. For example bridges located adjacent to signalised intersections will have far greater probability of queued traffic than highway bridges. For these reasons, the percentage of trucks that cross a bridge under queue conditions is site specific.

The number of trucks assumed to cross a bridge under queue conditions in published reports from previous investigations was reviewed. The site deemed most relevant was the West Gate Bridge in Melbourne. This site is considered to represent some of the heaviest examples of trucks under queue conditions, and as such is appropriate for establishing the design case. Cooper (2009) reported on a site-specific WiM analysis at this bridge, and concluded that 4% of traffic crossed the bridge in heavy congestion, and that vehicle spacing should be modelled with a 3m spacing between rear and front axles.

Cars act as spacers between trucks and reduce the average weight of queued traffic. The ratio of AADTT to AADT determines the average number of cars between trucks, and hence the average weight of queued traffic. Design should be based on the weight with an acceptably small probability of exceedance and not on the average weight; however, there is insufficient data to undertake a probabilistic analysis of the number of cars that should be allowed between trucks in queued traffic. This issue was addressed in the development of the bridge loading model for Eurocode (Bruls et al 1996), where the scenario of congested traffic or jam situations was based on an assumption of slow lane traffic without cars.

The assumption of no cars in the congested traffic was adopted in the queue scenarios in this WiM analysis. This is considered to be a realistic design case for short and medium-span bridges, but becomes increasingly conservative for longer-span bridges. The methodology is as follows:

- Each hour of records is treated as one queue. The entire hour worth of trucks is bunched together with a 3m gap between the rear axle of the leading vehicle and the front axle of the following vehicle. Light vehicles are ignored.

- The maximum daily value from the 24 queues is recorded for each load effect.

- It is assumed that 4% of trucks will cross the bridge under queued conditions. Therefore, each one day block is assumed to represent the maximum load effect for a 25-day block for return level calculations.

- The statistical analysis is run in the same way as for the observed flow analysis.

C3.1.3   1% queued flow scenario

Analysis assuming a 1% queue scenario was compared with the 4% queue scenario in order to determine the sensitivity of the results to the queue assumption. This scenario is identical to the 4% queue scenario, with the assumption that 1% of trucks cross the bridge under queued conditions. Therefore each one day block is assumed to represent the maximum load effect for a one hundred day block for return level calculations.

C3.2   Results

Tables C.7 through C.12 show the estimated return levels for 20-year return period and 2000 year return period for each load effect and span length, for the observed flow and queued analyses. The return level plots are shown in annex C1 for the observed flow analysis and annex C2 for the queued analyses. While most of the results are statistically significant fits to the GEV distributions, those that are poor fits are identified in the tables.

In cases where the GEV fit is poor, the return level extrapolations should not be relied upon.

Some of the return level plots appear to have systematic biases, with a number of observations with long return periods (in the top right of the plots) above the return level plot, even though the fit is considered good. The reason for this is that the great majority of observations have short return periods (in the bottom left of the plot), and therefore have much greater influence on the mathematical fit. It should be noted that most of the observations have a return level of less than 0.01 years (3.65 days) which do not appear on the plot at all. A visually better fit could be achieved by assigning more importance to the observations with long return periods; however, this would result in a mathematically poorer fit.

Possible reasons for these systematic biases include:

- The GEV analysis assumes that each observation is independent of the others. This assumption may occasionally be violated due to convoys of trucks with the same products and roughly controlled spacing.

- It is possible there are some minor systematic sensor calibration errors that the quality assurance process (see section C2.2) was unable to pick up.

Table C.7   Return levels for midspan moment, single span

M_m (kNm)	Span length (metres)						
	5	**10**	**20**	**40**	**60**	**80**	**100**
20yr, observed flow	253*	756	1720	4904	9888	14,497	19,114
20yr, queued flow 1%	N/A	N/A	1635	5500	11,841*	19,636	29,528
20yr queued flow 4%	N/A	N/A	1701	5607	11,994*	19,978	29,879
2000yr, observed flow	287*	970	1826	5179	10,223	14,682	19,227
2000yr, queued flow 1%	N/A	N/A	1823	5742	12,153*	20,416	30,244
2000yr queued flow 4%	N/A	N/A	1865	5772	12,181*	20,515	30,308

* poor fit to the GEV distribution (not statistically significant to a $p = 0.05$ level)

Table C.8 Return levels for shear at near support, single span

R_A (kNm)	Span length (metres)						
	5	10	20	40	60	80	100
20yr, observed flow	229	321*	359	539	660	733	827
20yr, queued flow 1%	N/A	N/A	375	594	817	1025	1235
20yr queued flow 4%	N/A	N/A	388	606	830	1039	1248
2000yr, observed flow	256	392*	373	566	667	738	835
2000yr, queued flow 1%	N/A	N/A	413	624	845	1055	1262
2000yr queued flow 4%	N/A	N/A	421	629	849	1058	1264

* poor fit to the GEV distribution (not statistically significant to a p = 0.05 level)

Table C.9 Return levels for shear at far support, single span

R_B (kN)	Span length (metres)						
	5	10	20	40	60	80	100
20yr, observed flow	222	321*	412	588	694	751	837
20yr, queued flow 1%	N/A	N/A	369	587	810	1027	1233
20yr queued flow 4%	N/A	N/A	382	598	823	1037	1247
2000yr, observed flow	249	391*	452	648	706	757	845
2000yr, queued flow 1%	N/A	N/A	405	612	840	1047	1262
2000yr queued flow 4%	N/A	N/A	413	616	843	1049	1265

* poor fit to the GEV distribution (not statistically significant to a p = 0.05 level)

Table C.10 Return levels for moment at central support, twin span

M_B (kNm)	Span length (metres)						
	5	10	20	40	60	80	100
20yr, observed flow	169	484	1235	3172	6649	9912	13,227
20yr, queued flow 1%	N/A	N/A	1397	4868	10,437	17,819	27,096
20yr queued flow 4%	N/A	N/A	1424	4933	10,579	18,002	27,337
2000yr, observed flow	185	526	1247	3175	6755	10,184	13,520
2000yr, queued flow 1%	N/A	N/A	1461	5005	10,732	18,170	27,628
2000yr queued flow 4%	N/A	N/A	1470	5019	10,759	18,194	27,664

* poor fit to the GEV distribution (not statistically significant to a p = 0.05 level)

Table C.11 Return levels for shear in near span at central support, twin span

V_{AB} (kN)	Span length (metres)						
	5	10	20	40	60	80	100
20yr, observed flow	241	341*	445	657	749	813	911
20yr, queued flow 1%	N/A	N/A	384	613	853	1077	1292
20yr queued flow 4%	N/A	N/A	399	624	868	1092	1307
2000yr, observed flow	272	406*	487	728	757	818	922
2000yr, queued flow 1%	N/A	N/A	426	640	887	1111	1323
2000yr queued flow 4%	N/A	N/A	436	643	892	1115	1326

* poor fit to the GEV distribution (not statistically significant to a p = 0.05 level)

Table C.12   Return levels for shear in far span at central support, twin span

V_{BC} (kN)	Span length (metres)						
	5	10	20	40	60	80	100
20yr, observed flow	250*	348*	398	602	726	800	904
20yr, queued flow 1%	N/A	N/A	428	699	964	1228	1475
20yr queued flow 4%	N/A	N/A	441	713	978	1245	1491
2000yr, observed flow	284*	414*	415	631	731	804	914
2000yr, queued flow 1%	N/A	N/A	462	732	995	1264	1506
2000yr queued flow 4%	N/A	N/A	468	737	998	1268	1508

* poor fit to the GEV distribution (not statistically significant to a p = 0.05 level)

C4   Summary

The Transport Agency provided the researchers with WiM data from five sites dating back to year 2000 for use in developing a new vehicle loading standard for road bridges in New Zealand. The WiM data was used to construct artificial queues of vehicles on bridges from which the loading event with a limiting probability of occurrence could be determined as the basis for the new vehicle loading.

From the five available sites, one was selected for detailed statistical analysis of the extreme values. Selection of the analysis site was based on the average and maximum shear and moment values extracted from a preliminary loading analysis, which included all lanes at all sites for 2006 to 2011. Average values for each load effect were found to be very similar across all sites; however, Drury lane 4 had higher maximum values. Being close to Auckland, Drury lane 4 also had a larger amount of traffic and particularly, heavy vehicles.

Load effects from single axles on simply supported spans and twin continuous spans (ie equal span lengths) for a range of span lengths were calculated, as follows:

- maximum mid span moment for a single span (M_m)

- maximum shear at near support for a single span (R_A)

- maximum shear at far support for a single span (R_B)

- maximum moment at central support for a twin continuous span (M_B)

- maximum shear in near span at central support for a twin continuous span (V_{AB})

- maximum shear in far span at central support for a twin continuous span (V_{BC}).

The load effects were determined for observed flow conditions (as observed at the WiM site) as well as two hypothetical queued scenarios for stationary loads. The queued scenarios were constructed by artificially reducing the gap between vehicle records to 3m (between the rear axle of the leading vehicle and the front axle of the following vehicle) and assuming that either 1% or 4% of heavy vehicles entered the site in queued conditions.

The extRemes (ismev) package for the R statistical language was used to perform a GEV analysis on each load effect for each span length.

Once the distribution parameters were fitted to the observed data, curves were extrapolated to predict the load effects for the range of span lengths for a 20 year return period and a 2000 year return period.

C5   References

Beca (2012) *Heavy vehicle data collection, analysis, and validation summary* rev A. Research report TAR 11/05.

Bruls, A, J Calgaro and H Mathieu (1996) ENV 1991 part3: the main models of traffic loads on road bridges: background studies. *IABSE Colloquium 74*, Delft, Netherlands, 1996.

Caprani, CC, EJ O'Brien and GJ McLachlan (2008) Characteristic traffic load effects from a mixture of loading events on short to medium span bridges. *Structural Safety 30*, no.5: 394–404.

Castillo, E (1988) *Extreme value theory in engineering*. San Diego: Academic Press.

Cooper, D (2009) Highway traffic load model for bridge design and assessment. *Austroads Bridge Conference,* Auckland, 2009.

Enright, B and EJ O'Brien (2011) *Cleaning weigh-in-motion data: techniques and recommendations*. International Society for Weigh-In-Motion. Accessed 17 October 2013. http://iswim.free.fr/doc/wim_data_cleaning_ie.pdf

New Zealand Transport Agency (2011) *Annual weigh-in-motion* (WiM) *report 2010*. Wellington.

Nowak, AS (1993) Live load model for highway bridges. *Structural Safety 13*, no.1-2: 53–66.

Wikipedia (2013a) Accessed 15 February 2013. http://en.wikipedia.org/wiki/Fisher%E2%80%93Tippett%E2%80%93Gnedenko_theorem

Wikipedia (2013b) Accessed 15 February 2013. http://en.wikipedia.org/wiki/Generalized_extreme_value_distribution

Annex C1: Observed flow analysis return level plots

Return level plot for shear at near support (R_A) 5m single span

Return level plot for shear at far support (R_B) 5m single span

Return level plot for moment at central support (M_B) 5m twin span

Return level plot for shear in near span at central support (V_{AB}) 5m twin span

Return level plot for shear in far span at central support (V_{ac}) 5m twin span

Return level plot for midspan moment (M_m) 10m single span

Return level plot for shear at near support (R_A) 10m single span

Return level plot for shear at far support (R_B) 10m single span

Return level plot for moment at central support (M_B) 10m twin span

Return level plot for shear in near span at central support (V_{AB}) 10m twin span

Return level plot for shear in far span at central support (V_{ilc}) 10m twin span

Return level plot for midspan moment (M_m) 20m single span

Return level plot for shear at near support (R_A) 20m single span

Return level plot for shear at far support (R_B) 20m single span

Return level plot for moment at central support (M_B) 20m twin span

Return level plot for shear in near span at central support (V_{AB}) 20m twin span

Return level plot for shear in far span at central support (V_{BC}) 20m twin span

Return level plot for midspan moment (M_m) 40m single span

Return level plot for moment at central support (M_B) 40m twin span

Return level plot for shear in near span at central support (V_{AB}) 40m twin span

Return level plot for shear in far span at central support (V_{BC}) 40m twin span

Return level plot for midspan moment (M_m) 60m single span

Return level plot for shear at near support (R_A) 60m single span

Return level plot for shear at far support (R_B) 60m single span

Return level plot for moment at central support (M_B) 60m twin span

Return level plot for shear in near span at central support (V_{AB}) 60m twin span

Return level plot for shear in far span at central support (V_{BC}) 60m twin span

Return level plot for midspan moment (M_m) 80m single span

Return level plot for shear at near support (R_A) 80m single span

Return level plot for shear at far support (R_B) 80m single span

Return level plot for moment at central support (M_B) 80m twin span

Return level plot for shear in near span at central support (V_{AB}) 80m twin span

Return level plot for shear in far span at central support (V_{ac}) 80m twin span

Return level plot for midspan moment (M_m) 100m single span

Return level plot for shear at near support (R_A) 100m single span

Return level plot for shear at far support (R_B) 100m single span

Return level plot for moment at central support (M_B) 100m twin span

Return level plot for shear in near span at central support (V_{AB}) 100m twin span

Return level plot for shear in far span at central support (V_{BC}) 100m twin span

Annex C2: Queued flow analysis return level plots

Return level plot midspan moment (M_m) 20m single span

Return level plot for shear at near support (R_A) 20m single span

Return level plot for shear at far support (R_B) 20m single span

Return level plot for moment at central support (M_B) 20m twin span

Return level plot for shear in near span at central support (V_{AB}) 20m twin span

Return level plot for shear at near support (R_A) 40m single span

Return level plot for shear at far support (R_B) 40m single span

Return level plot for shear in far span at central support (V_{BC}) 40m twin span

Return level plot for midspan moment (M_m) 60m single span

Return level plot for shear at near support (R_A) 60m single span

Return level plot for shear at far support (R_B) 60m single span

Return level plot for shear in far span at central support (V_{BC}) 60m twin span

Return level plot for midspan moment (M_m) 80m single span

Return level plot for shear at near support (R_A) 80m single span

Return level plot for shear at far support (R_B) 80m single span

Return level plot for moment at central support (M_B) 80m twin span

Return level plot for shear in near span at central support (V_AB) 80m twin span

Return level plot for shear in far span at central support (V_{BC}) 80m twin span

Return level plot for midspan moment (M_m) 100m single span

Return level plot for moment at central support (M_B) 100m twin span

Return level plot for shear in near span at central support (V_{AB}) 100m twin span

Return level plot for shear in far span at central support (V_{BC}) 100m twin span

Appendix D: Model development

D1 Approach to the model development

The primary purpose of this research was to determine a new vehicle loading standard for the design and evaluation of road bridges and other highway infrastructure in New Zealand. Implicit in this purpose was the necessity to decide if the existing vehicle loading standard should be changed or whether the status quo should remain. If a change was warranted, a decision had to be made about the form of the new vehicle loading standard.

Bridge loading standards around the world vary in the way they define the design vehicle loading, and how they prescribe the types of load to be designed for. After considering the outcome of the literature review, which reviewed six loading standards from around the world, the researchers formed the view that a new loading standard for New Zealand should be achieved by modifying either the existing *Bridge manual* (NZ Transport Agency 2013), or the Australian bridge design standard (AS5100). The basis for this view was that:

• Implementing a new loading standard that was not based on an existing standard would be inefficient, particularly with regard to the development of software that is very much a part of modern bridge design and assessment.

• If the existing *Bridge manual* could not be modified to meet the requirements of a new vehicle loading standard, modifying an international standard other than from Australia would be contrary to the high-level objective of harmonising standards between New Zealand and Australia.

Therefore the development of the loading model for a new vehicle loading standard looked first at possible modifications to the *Bridge manual*, and second at how the AS5100 could be modified for the purpose.

The development of a new vehicle loading standard model combined a 'top down' and a 'bottom up' approach. The top down approach considered the future freight need and likely configuration of vehicles to meet that need, and the bottom up approach was based on analysing the loading from current traffic. The implementation and outcomes from these two approaches are described in the following sections.

The loading model was developed for single and continuous bridges with spans up to 100m.

D1.1 Layout of this appendix

The top down approach is described in section D2. Refer to figure D.1 for the layout of section D2. The bottom up approach is described in section D3. Refer to figure D.2 for the layout of section D3.

Section D4 combines the findings from the bottom up and top down approaches and recommends the loading model.

Other vehicle loads and load factors are described in section D5, and the evaluation loading is described in section D6.

There are 10 annexes to this appendix (annexes D1 to D10).

Figure D.1   Layout of this appendix – section D2
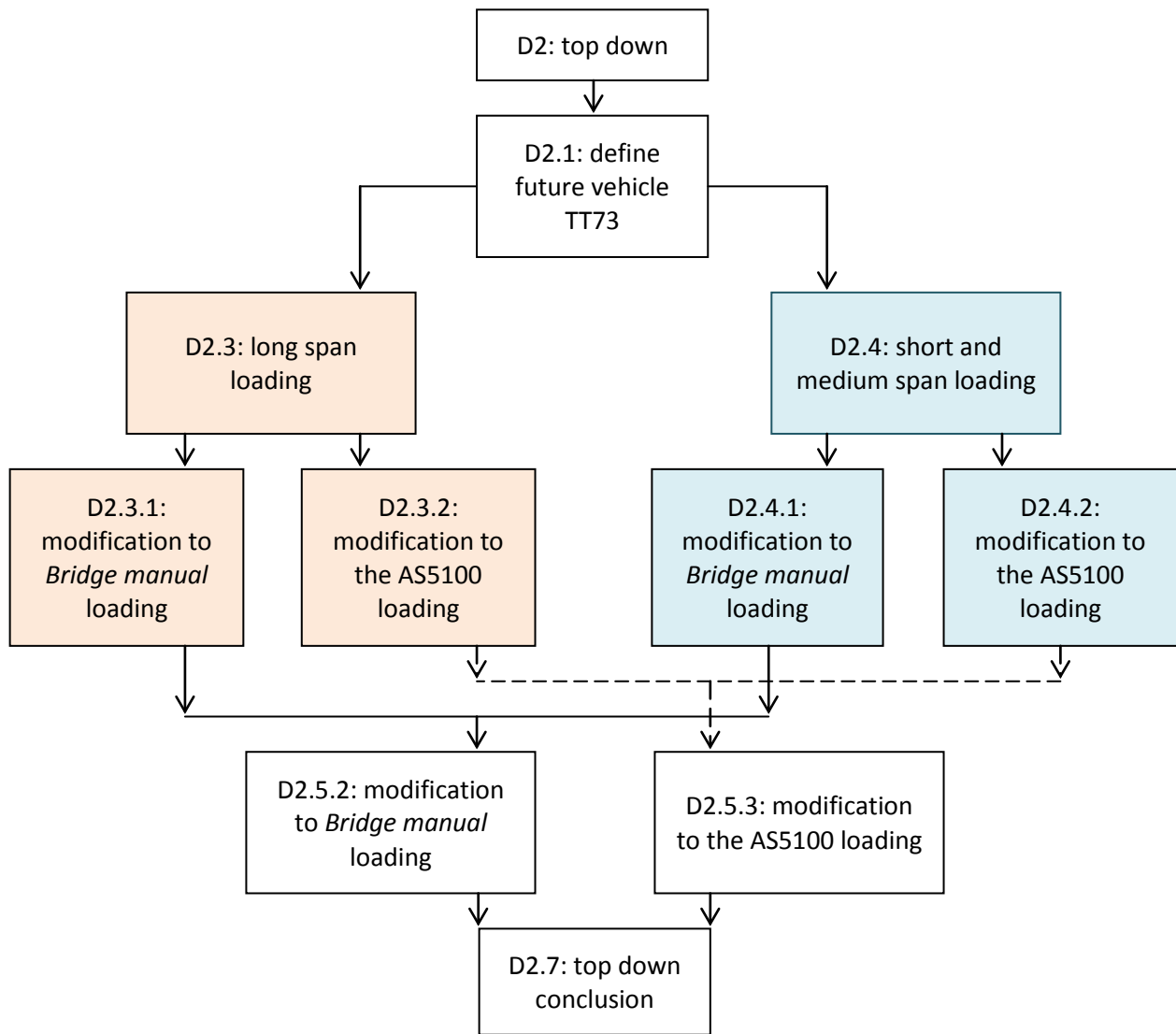
Figure D.2   Layout of this appendix – section D3

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┌──────────────┐
│ D3: bottom up │
└──────────────┘
        │
        ▼
┌──────────────┐
│   D3.3:      │                    ┌─────────────────┐
│ coincident   │───────────────────▶│ D3.10: actions  │
│ lane loading │                    │  from WiM data  │
└──────────────┘                    └─────────────────┘
        │                           ┌─────────────────┐
        │──────────────────────────▶│ D3.5: actions   │
        │                           │ from legally    │
        │                           │ loaded vehicles │
        │                           └─────────────────┘
        │    ┌─────────────────────┐
        │───▶│ D3.6: actions from  │
        │    │ vehicles proposed   │
        │    │ in AWLAG            │
        │    │ questionnaire       │
        │    └─────────────────────┘
        │    ┌─────────────────────┐
        │───▶│ D3.7: actions from  │
        │    │ NZ Transport        │
        │    │ Agency permit       │
        │    └─────────────────────┘
        │    ┌─────────────────────┐
        │───▶│ D3.8: actions from  │
             │ European cranes     │
             └─────────────────────┘
                      ┌─────────────────┐
                      │ D3.9: envelope  │
                      │ of actions from │
                      │ AWLAG, permits  │
                      │ and cranes      │
                      └─────────────────┘
                              ┌─────────────────┐
                              │   D3.11:        │
                              │ combined WiM    │
                              │ and vehicle     │
                              │ inputs          │
                              └─────────────────┘
          ┌─────────────────┐           ┌─────────────────┐
          │   D3.11.1:      │           │   D3.11.2:      │
          │ modification to │           │ modification to │
          │ *Bridge manual* │           │ the AS5100      │
          │ loading         │           │ loading         │
          └─────────────────┘           └─────────────────┘
                      ┌─────────────────┐
                      │ D3.12: bottom   │
                      │ up conclusion   │
                      └─────────────────┘
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D2   Top down approach

The basis of the top down approach to the development of a bridge loading model is to look at the development of future vehicles and develop a loading model that will cater for future vehicle loads. This is arguably a more rational approach than the alternative of assessing current loading, and adding a margin for future mass increases. The literature review identified the development of the Australian AS5100 standard as the only case where the design loading was unambiguously based upon the top down approach.

The objectives in developing the AS5100 were that the design loading model should provide an envelope of forces to ensure structures were covered for all present general access vehicles (including the emerging B-triples and the currently restricted triple road trains); present and projected special purpose vehicles; and projected potential future general access vehicles (ideally the loading model should cater for their ultimate evolution, but should aim to cover at least the next 50 years so that structures designed to this loading in 20 years would have a minimum life of 30 years).

Vehicle payloads were determined by dimension limits and freight density. The study sought to identify the density at which 90% of the freight task would be volume constrained, ie only 10% of the task would be constrained by mass limits. It was decided that the vehicle design loading should be based upon a freight density of 0.73t per cubic metre. This maximum density covered 80% of the freight task.

A range of vehicle configurations was considered in order to determine the range of design actions on bridges due to vehicles which were volume constrained with freight of that density. The critical vehicle for bridge loading, and the basis for the AS5100 SM1600 loading, was a B triple vehicle known as the BTS73. Loaded at 0.73t per cubic metre this has a gross combination mass of 152t and is shown in figure D.3.

Figure D.3   BTS73 triple road train (Heywood et al 2000)
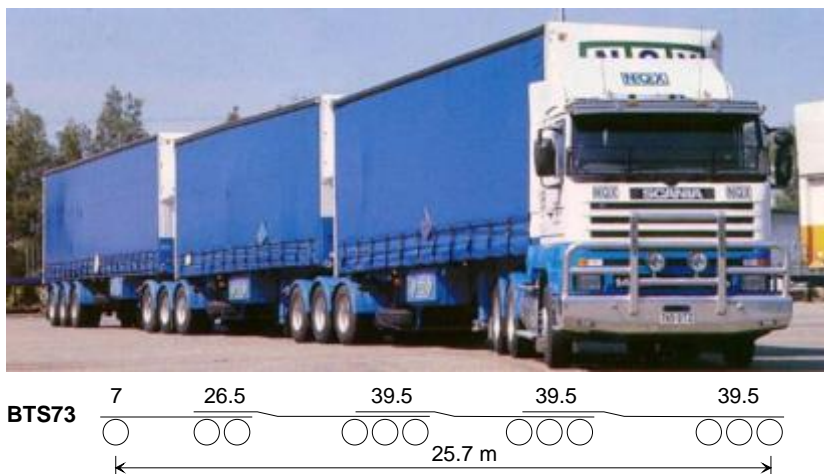
The triple road train is appropriate for long haul routes through Australia's sparsely populated regions; however, the researchers consider that vehicle is inappropriate to use for the development of a bridge loading model for New Zealand because the freight routes in New Zealand are not as long, and because the New Zealand topography means many roads have horizontal curves of relatively small radius which are not suited to road trains. Notwithstanding that, the freight density, and the general procedure that was used for the top down approach in Australia is considered an appropriate basis for the New Zealand situation.

D2.1 Projected potential future general access vehicles for New Zealand

The truck and full trailer combination is the dominant freight vehicle in New Zealand. The potential future general access vehicle was based upon this vehicle configuration with volume constrained loading at a density of 0.73t per cubic metre. This will be referred to as the TT73 vehicle.

The axle configuration adopted was based upon that provided by the Transport Agency as the pro-forma design for the truck and trailer HPMV design, as shown in figure D.4.

Figure D.4   TT73 truck and full trailer – potential future general access vehicle
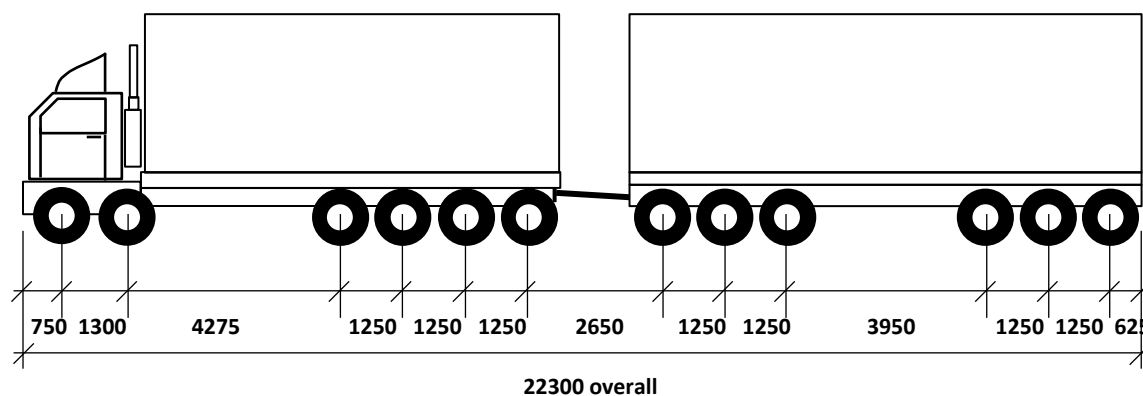
At volume constrained loading with a freight density of 0.73t per cubic metre the axle set loadings are:

1   dual steer   16t

2   quad   32t

3   tri-axles   27t

giving a gross combination mass of 102t (refer to the boxed text on the next page for a commentary on these axle loads). This vehicle will be used for the top down approach to the development of a bridge loading model.

The literature was reviewed to see what others had proposed as potential future heavy vehicles for New Zealand.

Sleath (1995) proposed eight vehicle configurations:

1   B train 7 axles   GCM 55.7t

2   B train 8 axles   GCM 55.7t

3   truck and trailer 6 axles   GCM 49.4t

4   truck and trailer 7 axles   GCM 53.8t

5   truck and trailer 8 axles   GCM 55t

6   logging jinker   GCM 42.4t

7   articulated truck   GCM 45.7t

8   A train logging truck   GCM 61.3t

TT73 vehicle axle loads

The axle loads for the TT73 vehicle were calculated on the following basis:

Truck

freight volume = 7.9(L) x 2.8(H) x 2.4(W) = 53.1m^3

freight weight = 53.1m^3 x 0.73t/m^3 = 38.8t

tare = 12t

gross weight = 50.8 t

weight on tandem steer = 0.25 x 38.8 + 0.75 x 12 = 18.7t

weight on quad axle set = 0.75 x 38.8 + 0.25 x 12 = 32.1t

Trailer

freight volume = 9.9(L) x 2.8(H) x 2.4(W) = 66.5m^3

freight weight = 66.5m^3 x 0.73t/m^3 = 48.6t

tare = 7 t

gross weight = 55.6 t

weight on each tri-axle set = 0.5 x 55.6 = 27.8 t

Limit the weight on tandem steer to 16 t, and round off weight on other axle sets to give the final weights:

tandem steer: 16t

quad axle set   32t

tri-axle sets   27t

total weight   102t

The overall axle spacing of the TT73 vehicle is 20.93 m. The mass intensity is 102 t/ 20.93 m = 4.9 t/m.

The overall axle spacing of the BTS73 vehicle used for the development of the AS5100 is 25.7 m and the weight is 152 t, giving a mass intensity of 5.9 t/m. Both vehicles are based on a freight density of 0.73 t/m3. The researchers reviewed every published (and some unpublished) papers describing the development of the Australian bridge design loading and did not find details of how the axle loads for the BTS73 vehicle were calculated. The difference in mass intensity between the TT73 and BTS73 vehicles could not be reconciled.

Wanty and Sleath (1998) proposed four vehicle configurations:

1   truck and trailer 8 axles   GCM 55t

2   articulated truck   GCM 45t

3   A train   GCM 50t

4   B-double   GCM 67t

Roberts and Heywood (2001) identified three critical vehicles from a range of potential configurations:

1   B double (B1233)   GCM 45.5t

2   B double (B1233)   GCM 47t

3   B double (B1233)   GCM 62t

Sweatman et al (2004) identified an articulated (A224) vehicle as a potential future vehicle:

1   articulated truck   GCM 50.8t

de Pont (2008) proposed a future logging truck:

1   truck and trailer   GCM 52t.

It is apparent from the gross combination masses listed that none of these proposed future vehicles envisaged the development of a road network over the next 50 to 100 years to carry a new generation of freight vehicles operating at optimised freight density. To illustrate this, the mass intensity of each vehicle was calculated, assuming vehicles were queued with a 5.5m headway between the rear axle of one vehicle and the front axle of the following vehicle. Studies conducted for the development of the AS5100 resulted in a 5.5m headway being selected for that loading model (Heywood et al 2000, figure 4) and the same headway was assumed for this work. These densities, expressed as t/m of loaded lane length are plotted in figure D.5. The graph also plots the TT73 potential future general access vehicle, and the mass intensity of a typical vehicle that complies with the 2010 Vehicle Dimensions and Mass Amendment Rule. The previously proposed potential future general access vehicles have mass intensities that are less than the rule allows. The TT73 vehicle, by contrast, represents a change in thinking about what a future general access vehicle could look like if it were not constrained by the current bridge network. This is the philosophy that should be applied when considering a top down approach to future general access vehicles.

Figure D.5   Comparison of the mass intensities of potential future general access vehicles
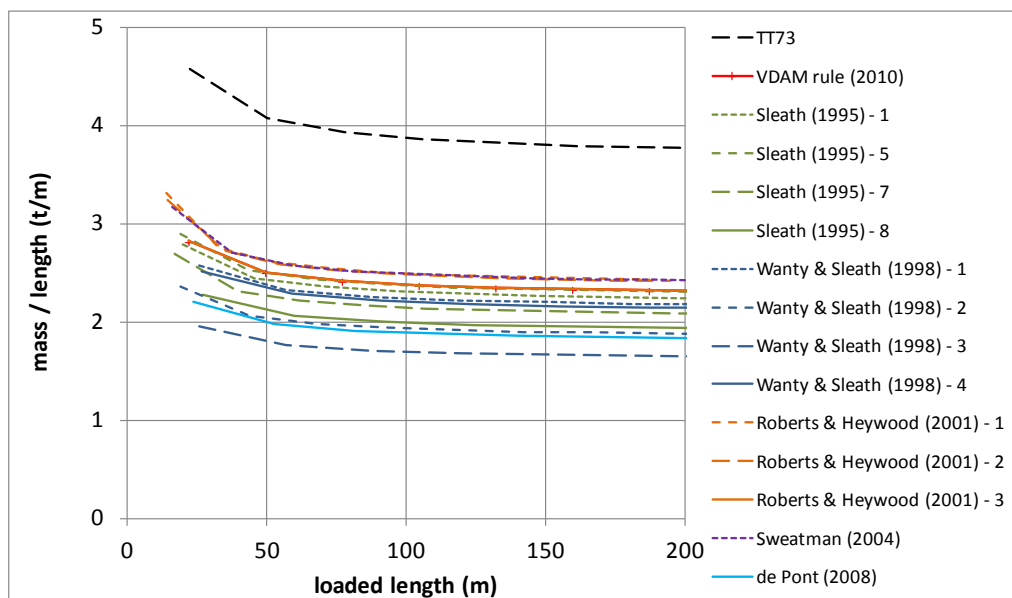
D2.2   Developing the design loading

The next stage of the model development using the top down approach was to develop a design model for new bridges that would allow for the TT73 vehicle.

The literature review referenced work by Heywood et al (2000) indicating that the average extreme daily actions induced in bridges by a single lane of traffic could be considered in three groups:

1   Long-span loading - queues of stationary vehicles.

2   Short and medium-span loading - legally loaded sets of axles or entire vehicles in which the distance between axle sets is at a minimum.

3   Local actions - slightly overloaded individual wheels, axles and axle sets.

The design model must address all three areas. In simple terms, long-span loading is represented by a queue of stationary vehicles at minimum headway, short and medium-span loading is represented by one or two moving vehicles in a lane, and local actions are represented by the design wheel and axle load. These span ranges will be considered in the next sections.

D2.3   Long-span loading – a queue of stationary TT73 vehicles

The values in this section do not include a dynamic load allowance (DLA).

The loading from a queue of stationary vehicles is dependent on the variation in loading from truck to truck, the number of cars between trucks, the headway between successive vehicles, and the frequency of queue formation. WiM studies can provide information on the likely distribution of these parameters for current traffic, but the top down approach requires assumptions to be made about values for each of these parameters at some future time, when the potential future general access vehicle goes into service.

In this research the difficulty in quantifying the loading from a queue of stationary vehicles was addressed by looking at how various international design codes model this loading.

The design loading from a range of international codes was calculated by applying a single lane of traffic over a range of loaded lengths up to 100m. These design loads are plotted in figure D.6.

It is apparent that below a loaded length of approximately 30m the design load varies irregularly due to the influence of individual axle loads. For loaded lengths beyond 30m the effect of individual axles is not apparent, and the distributed load component of the design loading takes over. It is this distributed load component that models the loading from a stationary queue of vehicles. To appreciate this better, the load was normalised by dividing by the load at 30m loaded length, for each design code. This is plotted in figure D.7. It is apparent that there is little variation in the normalised load between all of the codes beyond a loaded length of 30m.

Figure D.6   Load calculated from various international design codes

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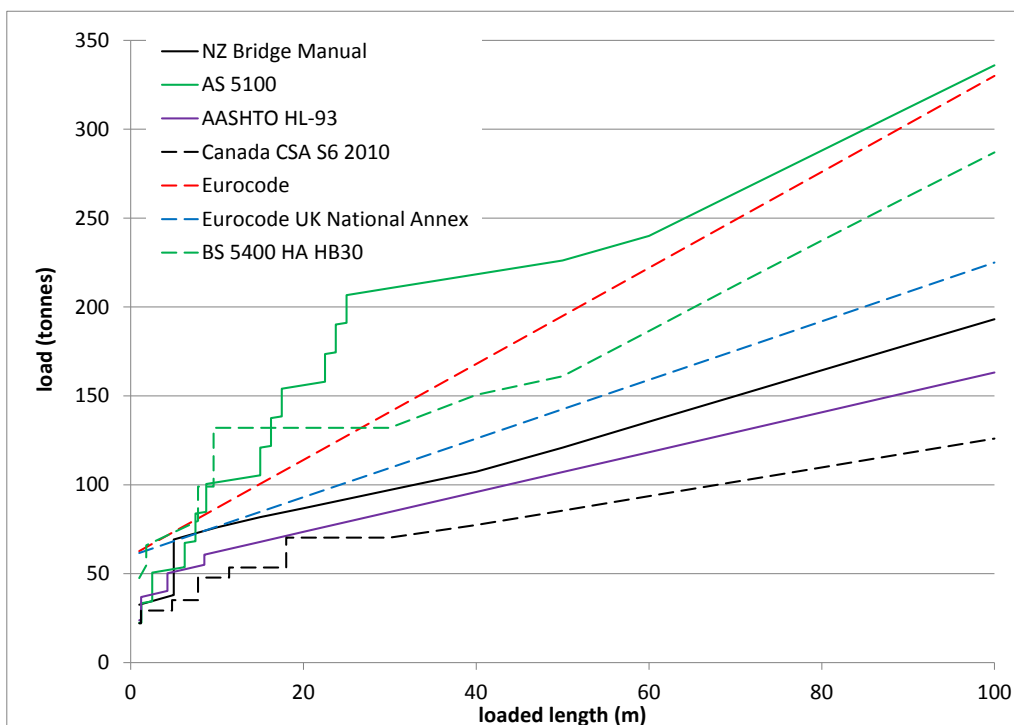
Figure D.7   Normalised load calculated from various international design codes
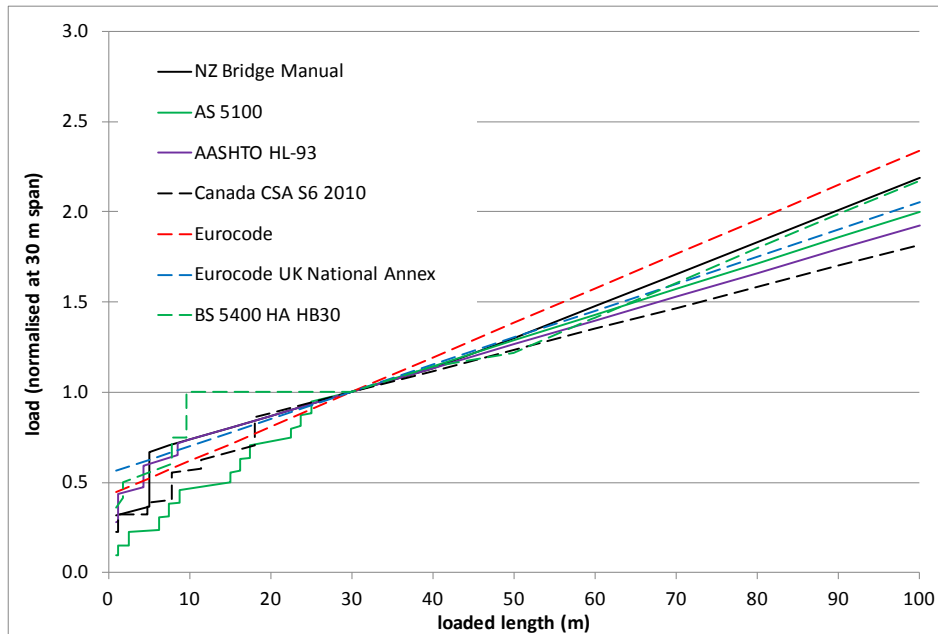
For the top down approach to the development of a design model, a mid-line was selected through this data. This matches the AS5100 curve, and also has the mathematical convenience of giving a normalised load at 30m of 1.0 and a normalised load at 100m of 2.0. For a loaded length in m the equation of the linear relationship is,

$$normalised \quad load_x = \frac{load_x}{load_{30}} = \frac{loaded \quad length}{70} + \frac{40}{70}$$

Multiplying by the loading on 30m loaded length gives an equation for the design load on any loaded length x as,

$$load_x = load_{30}\left[\frac{loaded \quad length}{70} + \frac{40}{70}\right]$$

For the TT73 potential future general access vehicle the maximum load on a 30m loaded length of single lane is 129t. This was determined by placing two vehicles in a lane with 5.5m headway between vehicles, and calculating the maximum value of the load on any 30m length.

Therefore the design model for the TT73 vehicle is represented by:

$$load(tonnes) = 129\left[\frac{loaded\ length}{70} + \frac{40}{70}\right]$$
$$= 1.84 \times loaded\ length + 73.7$$

or expressed in kN,

$$load(kN) = 18.1 \times loaded\ length + 723$$

which is a fixed load of 723kN and a udl of 18.1kN/m. This defines the design model for loaded lengths beyond that at which individual axle loads affect the loading (ie beyond 30m). It represents stationary, queued vehicles.

D2.3.1   Long-span loading – modification to *Bridge manual* loading

The HN-72 loading defined in the *Bridge manual* comprises a pair of 120kN axles and a udl of 10.5 kN/m. To match the loading from a queue of stationary TT73 vehicles the axle weights must be multiplied by 3.0 (to give a combined weight of 720kN) and the udl must be multiplied by 1.7 (to give a udl of 17.9kN/m). This loading is shown in figure D.8 and will be designated as 3.0/1.7_HN. The load intensity is plotted in figure D.10.

Figure D.8   Bridge manual HN loading to represent a queue of TT73 vehicles (3.0/1.7_HN)
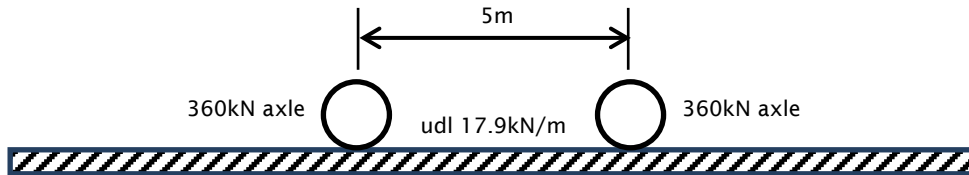
| Top down | long-span loading | modification to *Bridge manual* loading | 3.0/1.7_HN |

D2.3.2   Long-span loading – modification to the AS5100 loading

The AS5100 S1600 loading comprises four sets of 240kN tri-axles combined with a 24kN/m udl. To match the loading from a queue of stationary TT73 vehicles the tri-axle weights must be multiplied by 0.75 (to give a combined weight of 720kN) and the udl must also be multiplied by 0.75 (to give a udl of 18.0 kN/m). This loading is shown in figure D.9 and will be designated as 0.75_S1600. The load intensity is plotted in figure D.10.

Figure D.9   AS5100 S1600 loading factored to represent a queue of TT73 vehicles (0.75_S1600)
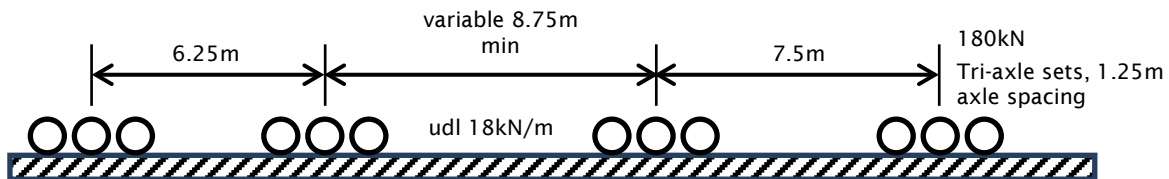
| Top down | long-span loading | modification to the AS5100 loading | 0.75_S1600 |

Figure D.10   Load intensity from the models representing a stationary queue of TT73 vehicles
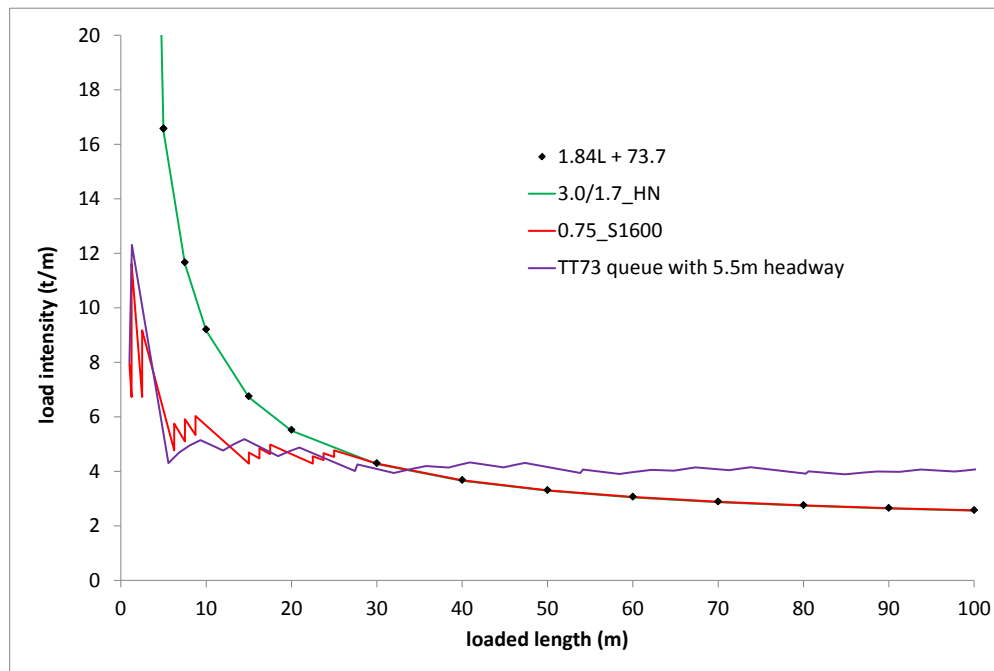
D2.3.3   Discussion – top down long-span loading

Both the *Bridge manual* HN-72 loading model and the AS5100 SM1600 loading model can be factored to represent the loading from a queue of stationary TT73 vehicles.

Because the *Bridge manual* HN-72 loading model only has two axle loads, the magnitude of each axle load required to model the 12 axle TT73 vehicle is high (360kN or 37t), and as such the loading model does not represent the physical loading.

The AS5100 S1600 loading model requires both the axle and udl components of the model to be adjusted by the same amount (0.75). This makes for a convenient adaptation of that loading model.

The recommended loading models for long spans based upon the top down approach are:

Top down	long-span loading	modification to *Bridge manual* loading	3.0/1.7_HN
Top down	long-span loading	modification to the AS5100 loading	0.75_S1600

D2.4   Short and medium-span loading – from TT73 vehicles

The values in this section do not include a DLA.

Short and medium-span loading is represented by legally loaded sets of axles or entire vehicles in which the distance between axle sets is at a minimum. The actual axle spacing and mass must be taken into account. Development of this part of the loading model was approached in two ways:

- A linear equation was fitted to the axle spacing and mass schedule (ASMS) for the TT73 vehicle. This is plotted in figure D.11.

- The design actions (moments and shears) from one or two TT73 vehicles with a minimum headway of 5.5m were calculated for single and twin continuous spans up to 30m. Beyond 30m the design loading transitions to the long-span loading case were represented by queues of stationary vehicles. The axle

loads from the second TT73 vehicle were factored by 0.8 to allow for the reduced probability of the two vehicles being fully loaded. The factor of 0.8 is consistent with the accompanying lane factor in the AS5100 for the case of two lanes loaded. For the twin continuous span case the headway was varied (with a minimum of 5.5m) to obtain the maximum hogging moment.

These two approaches were used to develop factored versions of the *Bridge manual* HN-72 loading and the AS5100 loading, for short and medium spans.

D2.4.1 Short and medium-span loading – modification to *Bridge manual* loading

The first approach to developing a factored version of the *Bridge manual* HN-72 loading for short and medium spans was to consider the axle spacing and mass schedule (ASMS). The ASMS graph for a single TT73 vehicle is shown in figure D.11.

Figure D.11 Axle spacing and mass graph for a single TT73 vehicle
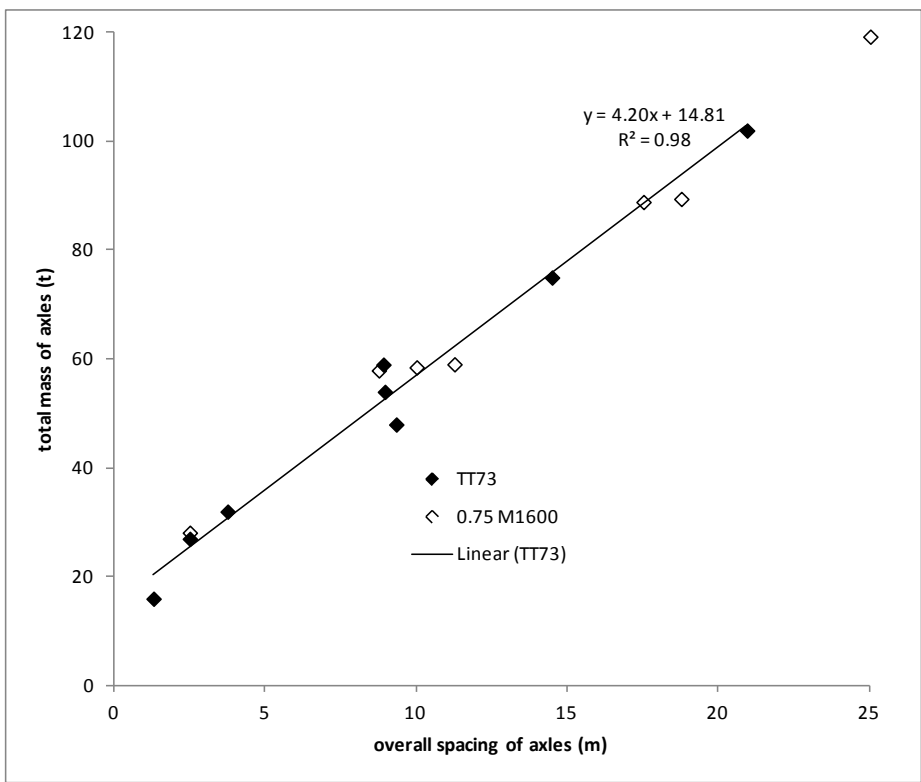
The linear fit in figure D.11 indicates that the TT73 is best represented by a fixed load of 14.81t (145 kN) and a udl of 4.2t/metre (41kN/m). Therefore the HN-72 axle loading must be factored by 0.61 (to give a combined weight of 146kN) and the udl must be factored by 3.9 (to give a udl of 41 kN/m). This loading is shown in figure D.12 and will be designated as 0.61/3.9_HN.

Figure D.12 *Bridge manual* HN-72 loading factored to represent the TT73 vehicle ASMS (0.61/3.9_HN)
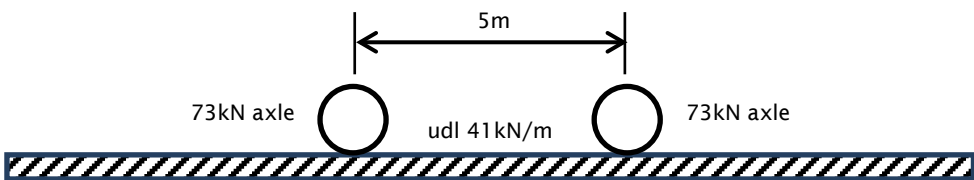
The second approach to developing a factored version of the *Bridge manual* HN-72 loading for short and medium spans was to consider the design actions from one or two TT73 vehicles in a lane (with 5.5m minimum headway and a 0.8 factor applied to the second vehicle). The HN-72 loading model was factored to provide a 'best fit' to each of maximum sagging moment, maximum shear (single spans) and maximum hogging moment due to the TT73 vehicles traversing a range of simple and twin continuous spans up to 30m.

The factors are shown in table D.1. For practical reasons in the implementation of a design loading, it is not acceptable to have separate factors for each design action, and therefore a single 'average' factor must be chosen. This cannot be a good fit for all design actions. A factor of 2.0 was selected and the loading of 2.0_HN-72 is shown in figure D.13.

Table D.1   Factors for the HN-72 loading to 'best fit' the design actions from TT73 vehicles

Design action	Factor
Sagging moment	1.70
Shear	1.57
Hogging moment	2.14
Adopted value	**2.0**

Figure D.13   *Bridge manual* HN-72 loading factored to represent the design actions from TT73 vehicles (2.0_HN)

It is apparent from figures D.12 and D.13 that the two approaches have yielded different factors for the HN-72 loading. In the interest of simplicity, the second approach with a single factor of 2.0 for both the axle load and the udl (ie figure D.13) was adopted.

Top down	short/medium-span loading modification to the *Bridge manual* loading	2.0_HN

D2.4.2   Short and medium-span loading – modification to the AS5100 loading

The first approach to developing a factored version of the AS5100 loading for short and medium spans was to consider the axle spacing and mass schedule (ASMS). The AS5100 M1600 loading model comprises four sets of tri-axles with an overall length of 25m, combined with a udl. The spread of axles means that there is a range of axle load and udl combinations in the M1600 loading model that could adequately represent the ASMS of a single TT73 vehicle. Applying a factor of 0.75 to the M1600 loading gave a satisfactory fit (the optimised factor was 0.736). This loading will be designated as 0.75_M1600. The ASMS for this is plotted in figure D.11 and the loading is shown in figure D.14.

Figure D.14   AS5100 M1600 loading factored to represent the TT73 vehicle ASMS (0.75_M1600)
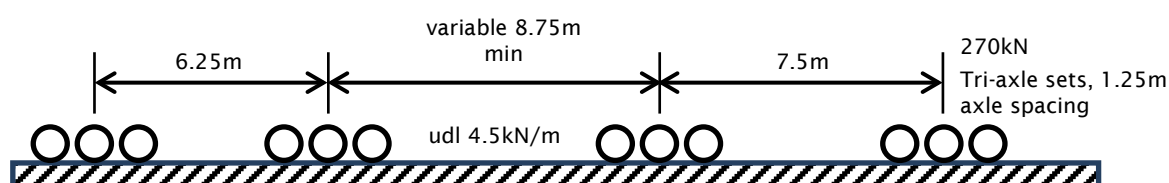
The second approach to developing a factored version of the AS5100 loading for short and medium spans was to consider the design actions from one or two TT73 vehicles in a lane (with 5.5m minimum headway and a 0.8 factor applied to the second vehicle). 0.80_M1600 loading provides an acceptable fit to the design actions due to the TT73 vehicles. For the continuous span case, the centre axle spacing in the M1600 model was varied in order to determine the maximum hogging moment, with a minimum spacing of 8.75m, consistent with the requirements in the AS5100. The loading is shown in figure D.15. The two approaches have yielded different factors for the M1600 loading, and the higher factor of 0.80_M1600 was adopted.

Figure D.15   AS5100 M1600 loading factored to represent the design actions from TT73 vehicles (0.80_M1600)
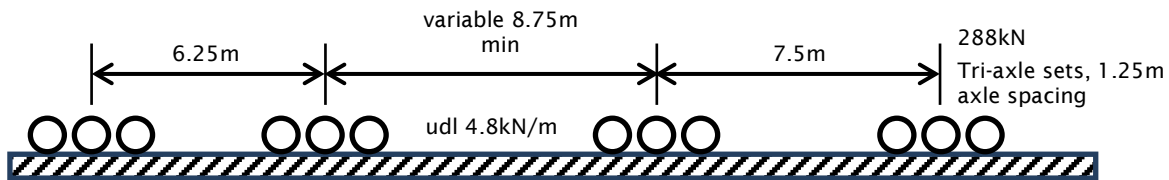
| Top down | short/medium-span loading modification to the AS5100 loading | 0.80_M1600 |

D2.4.3   Discussion – top down short and medium-span loading

The recommended loading models for short and medium spans based upon the top down approach are:

| Top down | short/medium-span loading modification to the *Bridge manual* loading | 2.0_HN |
| Top down | short/medium-span loading modification to the AS5100 loading | 0.80_M1600 |

The results for sagging moment, shear, and hogging moment are plotted in figures D.16 to D.20 and the variation of the recommended loading models from the actions due to the TT73 vehicle are summarised in table D.2. The values do not include a DLA. The results are presented over a span range of 1m to 50m (the short and medium-span range).

Figure D.16   Top down short/medium-span modification to *Bridge manual* loading – sagging moment
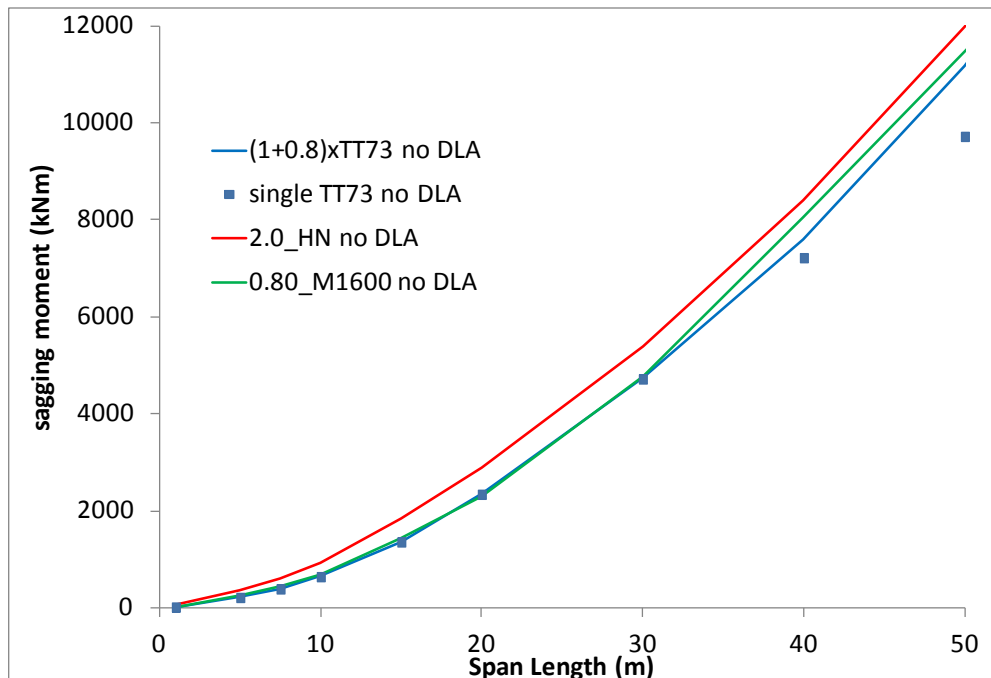
Figure D.17   Top down short/medium-span modification to the *Bridge manual* loading – sagging moment
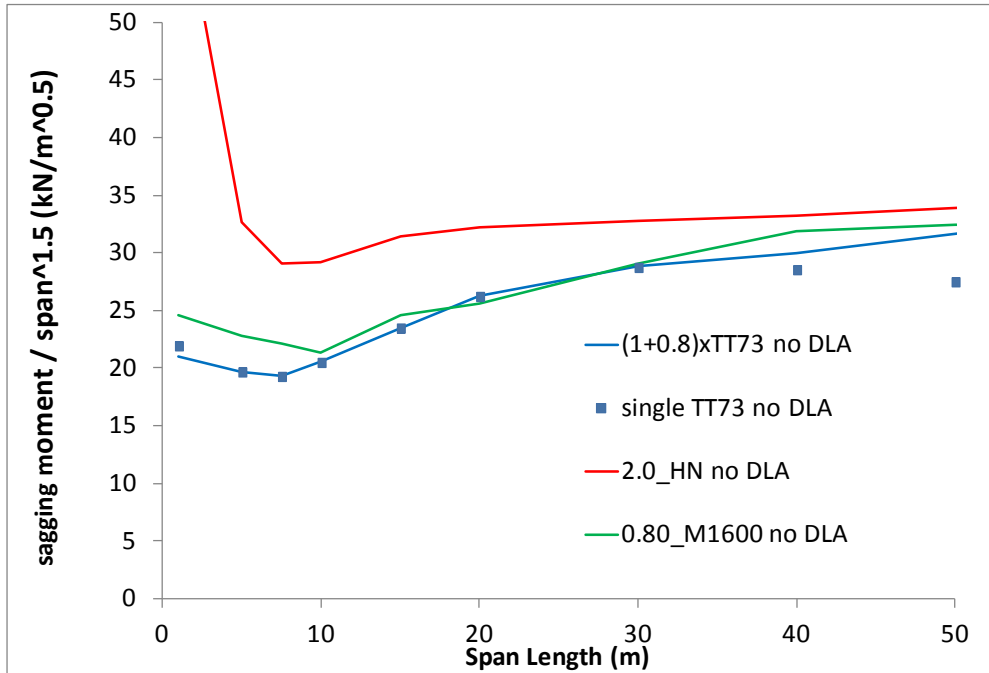
Figure D.18   Top down short/medium-span modification to the *Bridge manual* loading – hogging moment
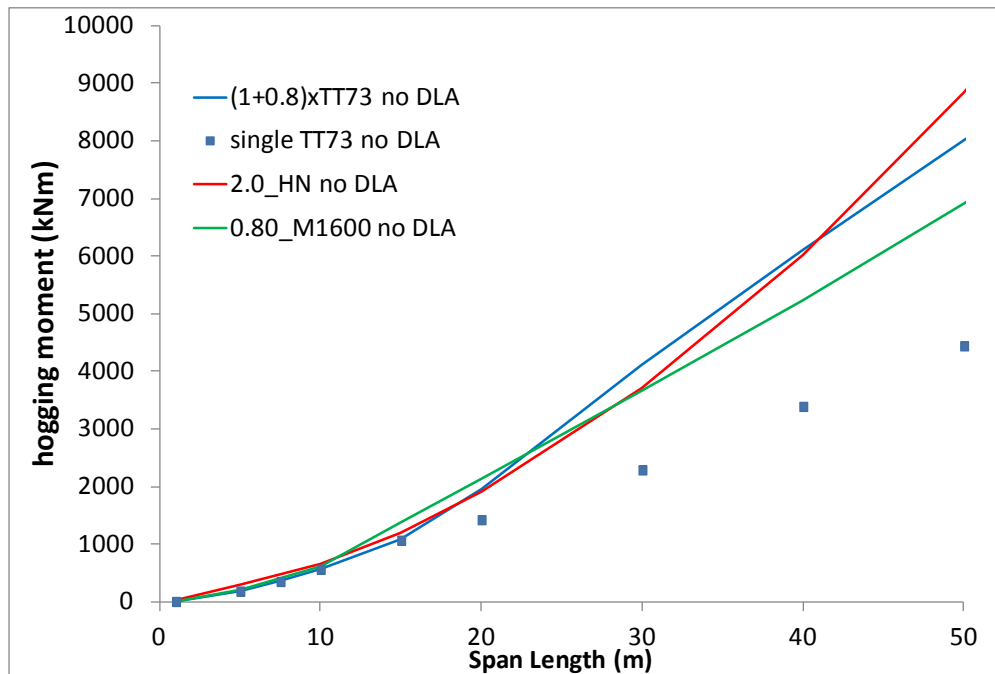
Figure D.19   Top down short/medium-span modification to the *Bridge manual* loading – hogging moment
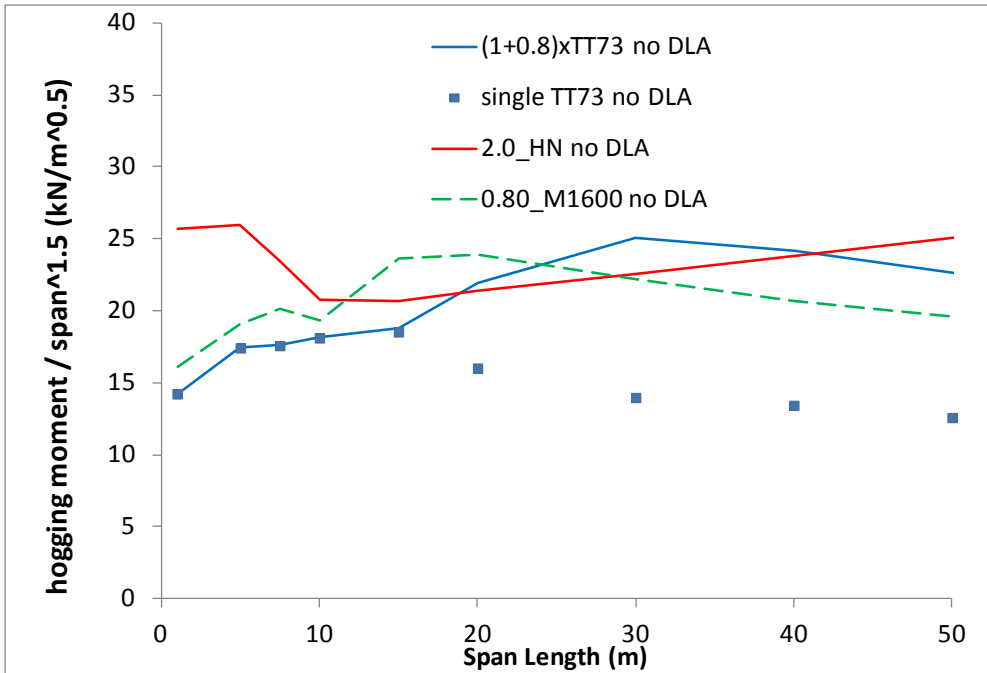
Figure D.20   Top down short/medium-span modification to the *Bridge manual* loading – shear
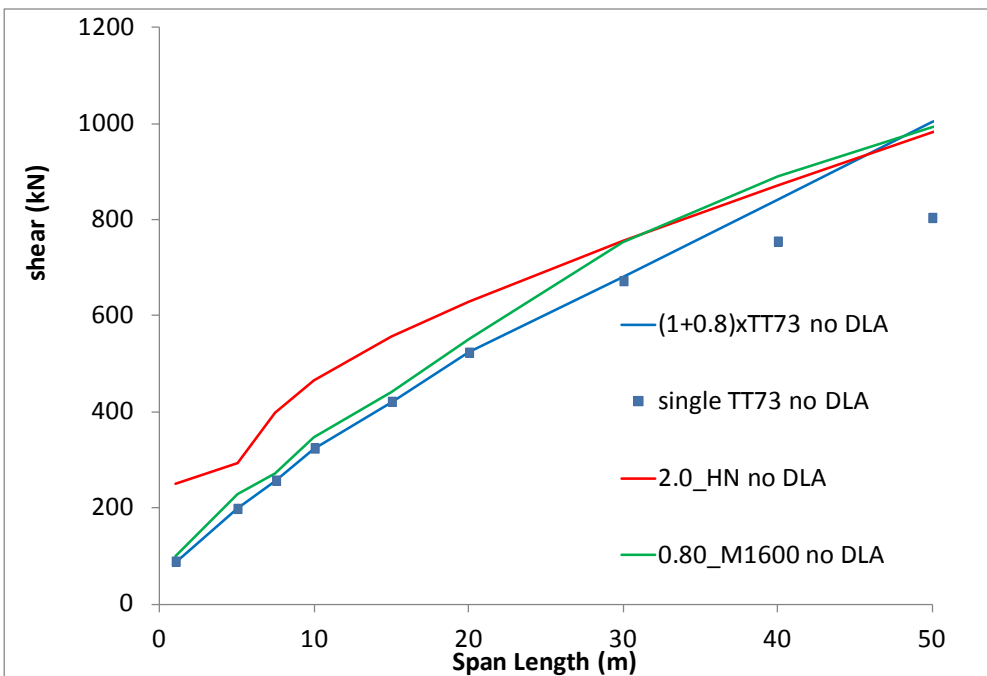
Table D.3   **Ratio of recommended model to the TT73 actions (top down short/medium span) – no DLA**

	Span	2.0_HN/TT73			0.8_M1600/TT73		
	Metres	Sagging moment	Shear	Hogging moment	Sagging moment	Shear	Hogging moment
Ratio of recommended model to a single TT73 vehicle	1	2.85	2.84	1.81	1.12	1.12	1.13
	5	1.66	1.48	1.49	1.16	1.15	1.10
	7.5	1.51	1.55	1.33	1.14	1.06	1.15
	10	1.42	1.44	1.15	1.04	1.08	1.07
	15	1.34	1.32	1.11	1.05	1.05	1.28
	20	1.23	1.20	1.33	**0.97**	1.06	1.49
	30	1.14	1.12	1.61	1.01	1.12	1.59
	40	1.16	1.15	1.77	1.12	1.18	1.54
	50	1.23	1.22	1.99	1.18	1.23	1.55
Ratio of recommended model to (1+0.8) TT73 vehicles	1	2.85	2.84	1.81	1.12	1.12	1.13
	5	1.66	1.48	1.49	1.16	1.15	1.09
	7.5	1.51	1.55	1.33	1.14	1.06	1.15
	10	1.42	1.44	1.15	1.04	1.08	1.07
	15	1.34	1.32	1.10	1.05	1.05	1.26
	20	1.23	1.20	**0.97**	**0.97**	1.06	1.09
	30	1.14	1.11	**0.90**	1.01	1.11	**0.89**
	40	1.11	1.03	**0.98**	1.06	1.06	**0.86**
	50	1.07	**0.98**	1.11	1.03	**0.99**	**0.86**

D2.5   Combining the long, short and medium-span loading

To cover the full range of loading up to 100m span it is necessary to combine the long span loading with the short and medium span loading. In the following sections combined loading models are first developed based on a modification to the *Bridge manual* loading, and then based on a modification to the AS5100 loading.

D2.5.1   Application of a dynamic load allowance

As discussed in the literature review (section A10.5) there is a common trend for the DLA to reduce towards zero for longer span bridges. As also discussed, the DLA is treated differently in the *Bridge manual* and in the Australian bridge design standard AS5100.

In the *Bridge manual* the DLA is a function of the span, reducing from 0.3 at 1m span down to 0.11 at 100m span. If the same value of DLA is applied to both the *Bridge manual* design vehicle loading and the TT73 vehicle loading, the ratio between the design vehicle load actions (moments, shears) and the TT73 load actions is unchanged with or without the DLA included. In the results presented below for a proposed modification to the *Bridge manual* loading this DLA has been included. This is done to be consistent with the results presented for a proposed modification to the AS5100 loading, which is discussed next.

In the AS5100 the DLA is treated quite differently. The AS5100 comprises two design vehicles. The M1600 vehicle represents the loading on short and medium spans and includes a DLA of 0.3. The S1600 vehicle represents the queued loading on long spans and has a DLA of 0.0. The actions from both vehicles must be

calculated and the greatest action controls. When comparing the load actions from the TT73 vehicle with the AS5100 design load actions it is necessary to apply a DLA of 0.3 to the TT73 vehicle on short to medium spans (for comparison with M1600 including DLA of 0.3), but no DLA on long spans (for comparison with S1600 with no DLA). Applying a DLA of 0.3 to the TT73 loading across the entire span range is conservative at long spans, because the DLA should reduce to zero. The dilemma that arises is how to transition the DLA applied to the TT73 vehicle as the span increases. One approach is to apply the DLA of 0.3 on short and medium spans only (say to 50m) and then no DLA for spans beyond 50m. This creates a step in the magnitude of the design load actions (at 50m). An alternative is to apply a graduated DLA which varies from 0.3 at short spans to 0.1, say, at 100m span. The latter approach has the advantage of avoiding a step in the magnitude of the design load actions, and is the approach that has been adopted in this research to enable a comparison to be made between the AS5100 vehicle design load actions and the TT73 vehicle load actions across the complete span range. In the absence of any definitive guidance on a suitable model for a graduated DLA, the DLA provided in the *Bridge manual* has been used for this purpose.

D2.5.2   Top down – modification to *Bridge manual* loading

The loading models determined from sections D2.3 and D2.4 are:

- long-span loading: 3.0/1.7_HN

- short and medium-span loading: 2.0_HN.

Figures D.20 to D.24 plot the sagging moment, hogging moment and shear actions from the TT73 vehicle and from these loading models. Comments on the plots are provided in table D.4.

In summary, 2.0_HN-72 is recommended as the modification to the *Bridge manual* loading, based upon the top down approach.

Top down	long/short/medium-span loading modification to the *Bridge manual* loading	2.0_HN-72

Table D.4   Modification to the *Bridge manual* loading – plots for sagging moment, hogging moment and shear

Design action	Comment
Sagging moment	The loading of 3.0/1.7_HN was developed for long-span loading from a queue of TT73 vehicles; however, the loading of 2.0_HN which was developed for the short to medium-span loading provides a better fit to the design actions over the entire span range and is recommended.
Hogging moment	Hogging moments in the span range of 20m to 40m are underestimated by the short and medium-span loading model by a maximum of 3%. This is considered acceptable.
Shear	Shears in the span range of 50m to 70m are underestimated by the short and medium span loading model by a maximum of 2%. This is considered acceptable.

Figure D.20   Modification to the *Bridge manual* loading – sagging moment
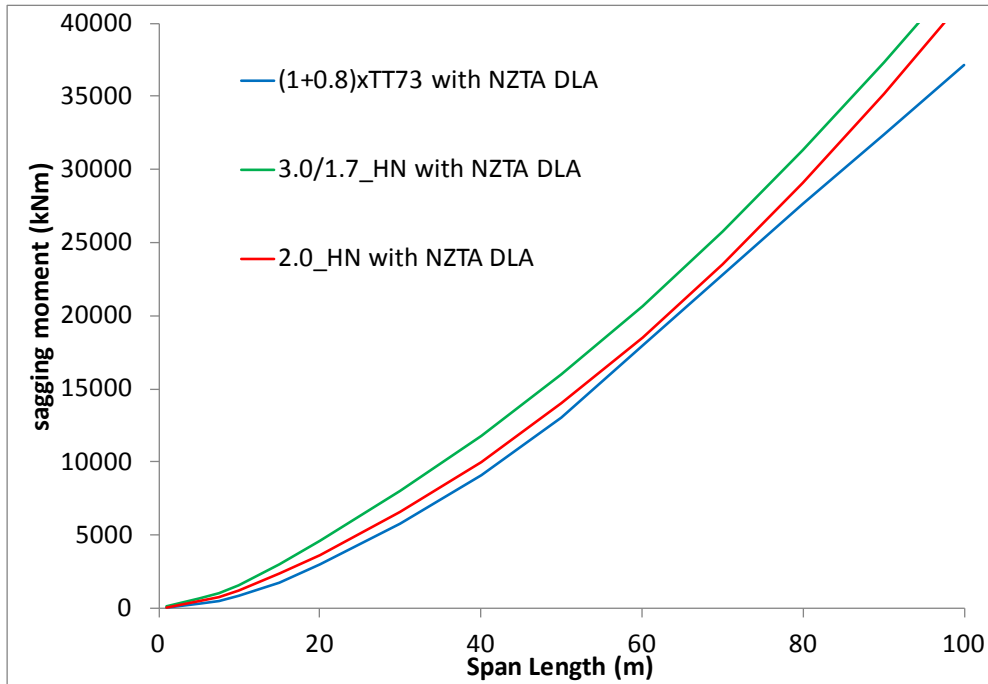
Figure D.21   Modification to the *Bridge manual* loading – sagging moment
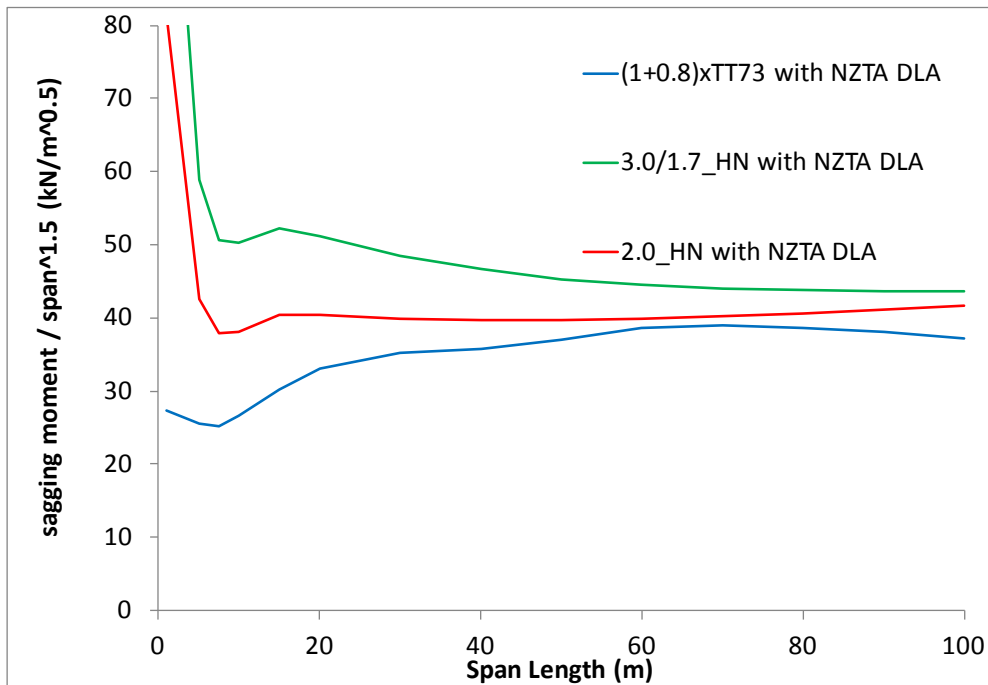
Figure D.22   Modification to the *Bridge manual* loading – hogging moment
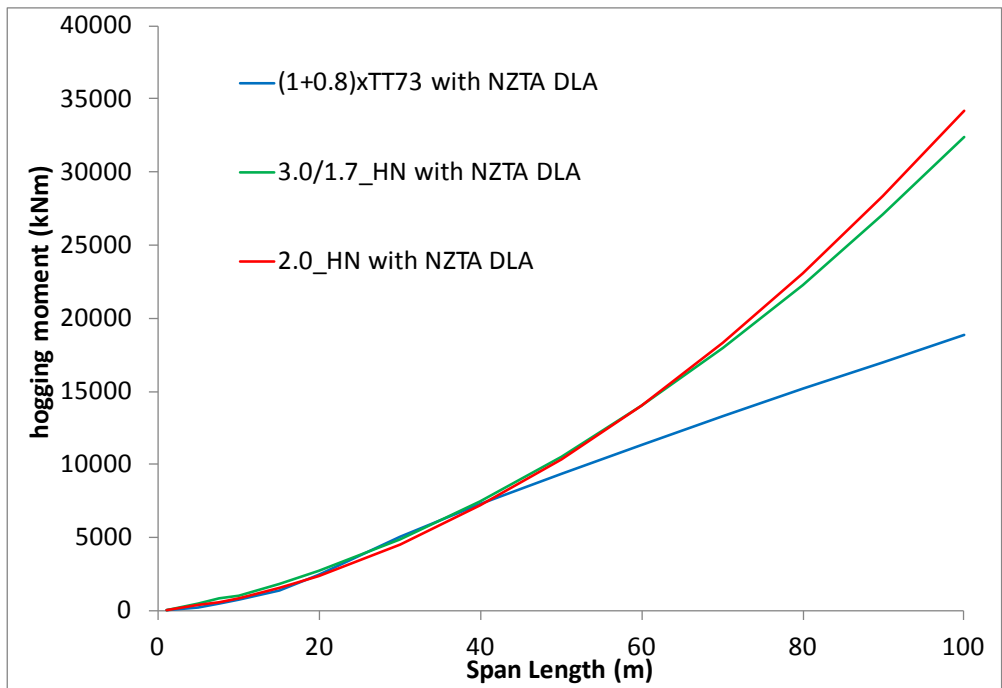
Figure D.23   Modification to the *Bridge manual* loading – hogging moment
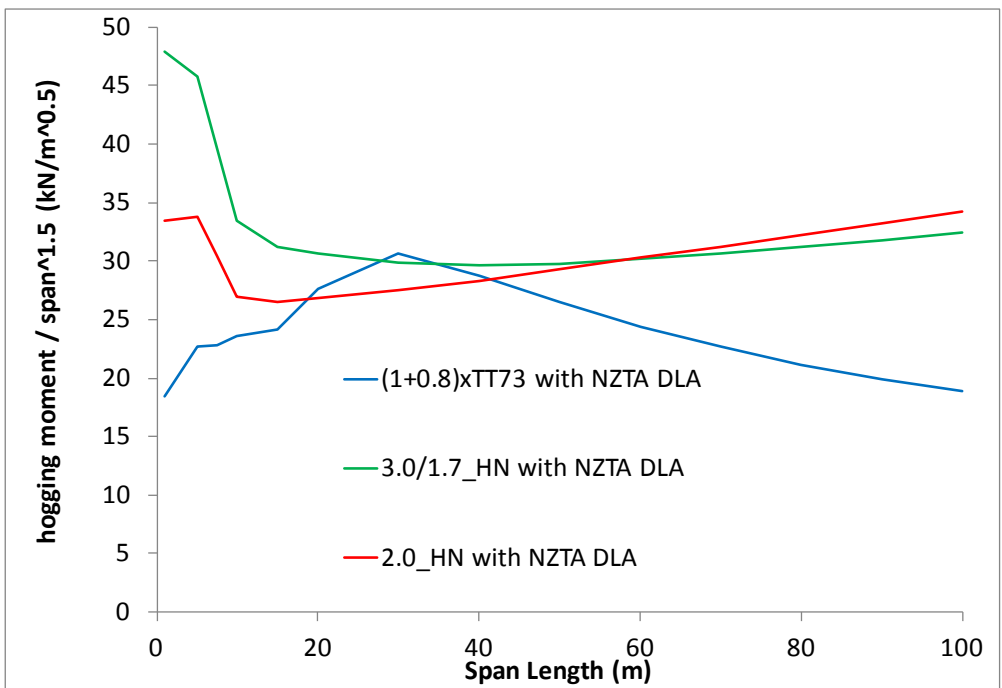
Figure D.24   Modification to the *Bridge manual* loading – shear
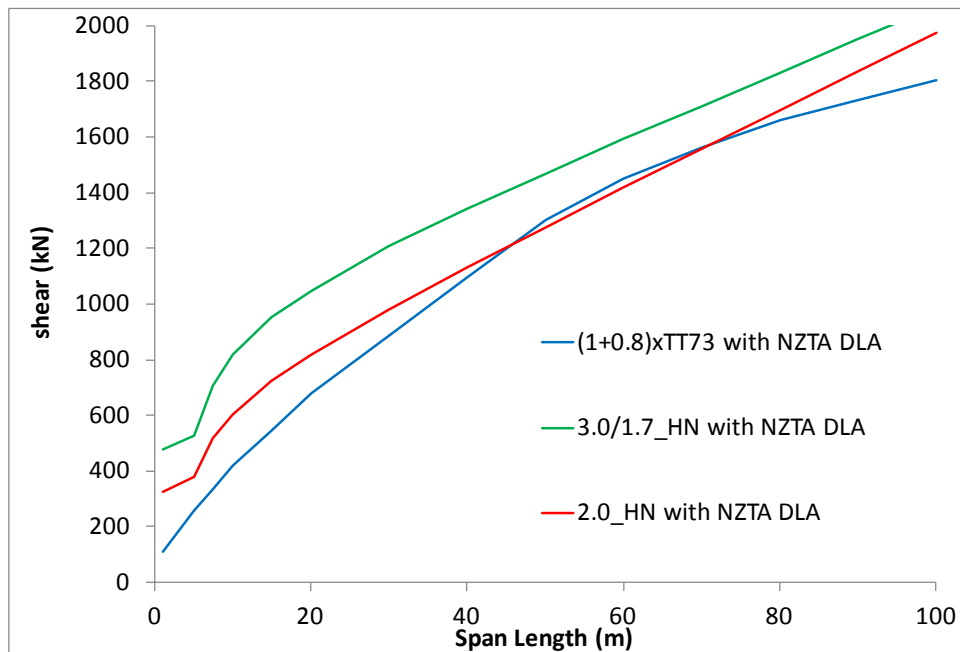
D2.5.3   Top down – modification to the AS5100 loading

The loading models determined from sections D2.3 and D2.4 are:

- Long-span loading: 0.75_S1600
- short and medium-span loading: 0.80_M1600.

The approach of the AS5100 loading is that the most severe action from S1600 and M1600 loading controls. To simplify the adaptation of AS5100 it is recommended that the same factor should be applied to both the S1600 and the M1600 loading, and that factor should be 0.80. Figures D.25 to D.29 plot the sagging moment, hogging moment and shear design actions from the TT73 vehicle, together with the actions from the 0.80_S1600 and 0.80_M1600 loading models.

Specific comments on the plots are provided in table D.5.

In summary the combination of 0.80_S1600 and 0.80_M1600 (known as 0.8_SM1600) provides an acceptable design model over the span range, based upon the top down approach.

| Top down | long/short/medium-span loading | modification to the AS5100 loading | 0.80_SM1600 |

Table D.5   Modification to the AS5100 – plots for sagging moment, hogging moment and shear

Design action	Comment
Sagging moment	0.80_SM1600 provides a good model for the transition from short and medium to long-span loading. 0.80_M1600 controls for spans to approximately 90m, with 0.80_S1600 being the critical loading for 90m to 100m spans.
Hogging moment	0.80_M1600 controls for spans to approximately 50m, with 0.80_S1600 being the critical loading for 50m to 100m spans. Hogging moments in the span range of 30m to 50m are underestimated by a maximum of 6%. This is considered acceptable. The maximum hogging moment occurs when a TT73 vehicle is followed by 0.8 of a TT73 vehicle at the unique value of headway required to maximise the hogging moment. Coincidence of these three requirements is extremely unlikely.
Shear	0.80_M1600 controls over the entire span range. At longer spans the loading model underestimates the shears by up to 9%. This is largely due to the application of the DLA – for shear the *Bridge manual* applies a DLA of 0.3 which does not decrease at long spans.

Figure D.25   Modification to the AS5100 loading – sagging moment
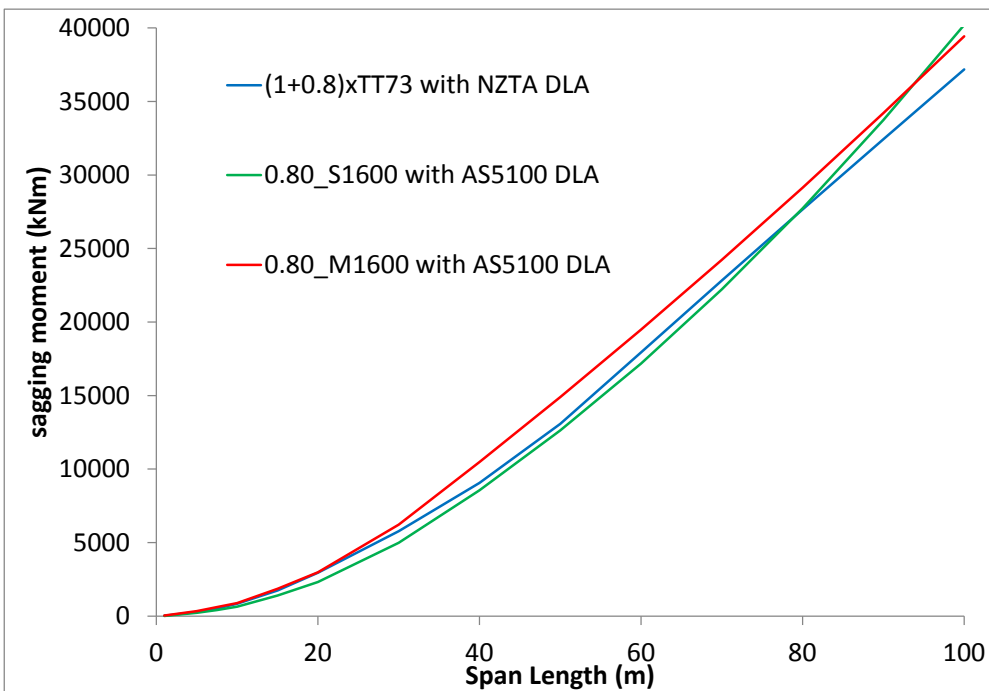
Figure D.26   Modification to the AS5100 loading – sagging moment
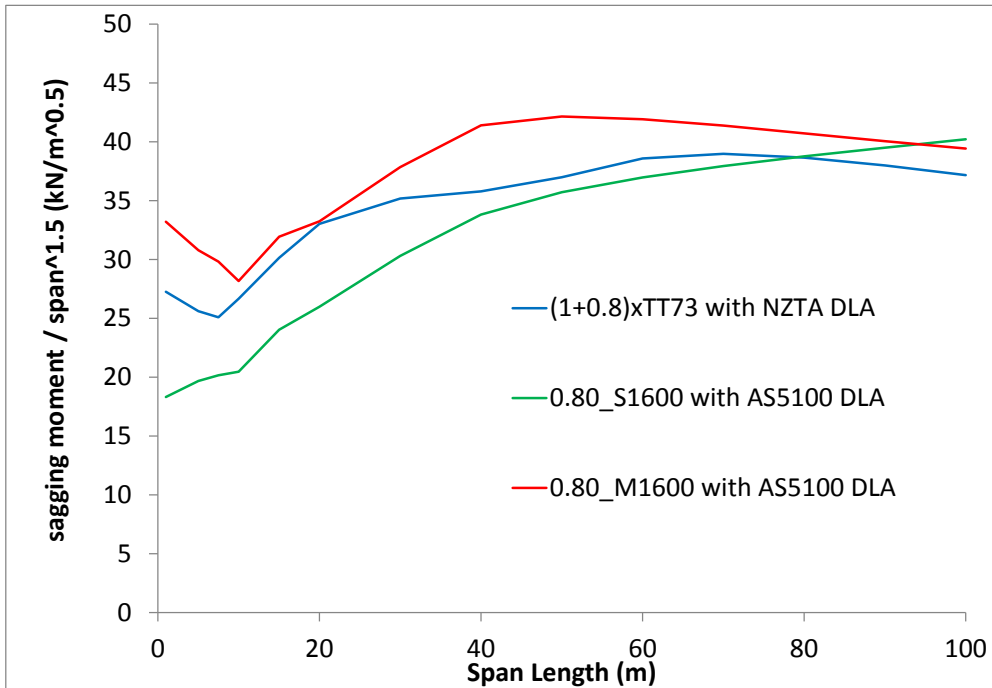
Figure D.27   Modification to the AS5100 loading – hogging moment
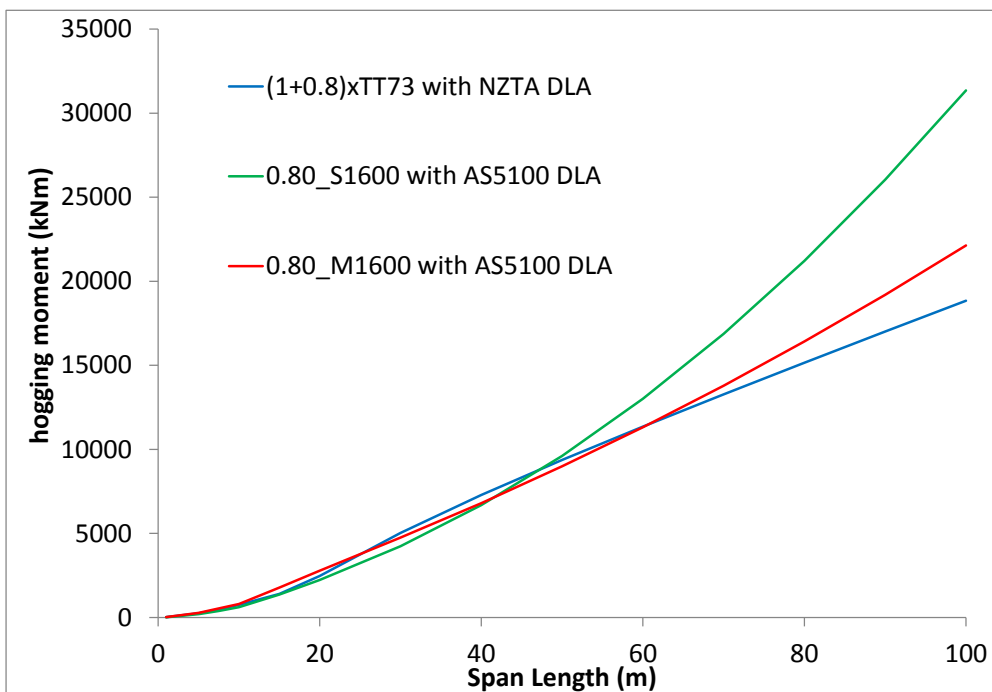
Figure D.28   Modification to the AS5100 loading – hogging moment
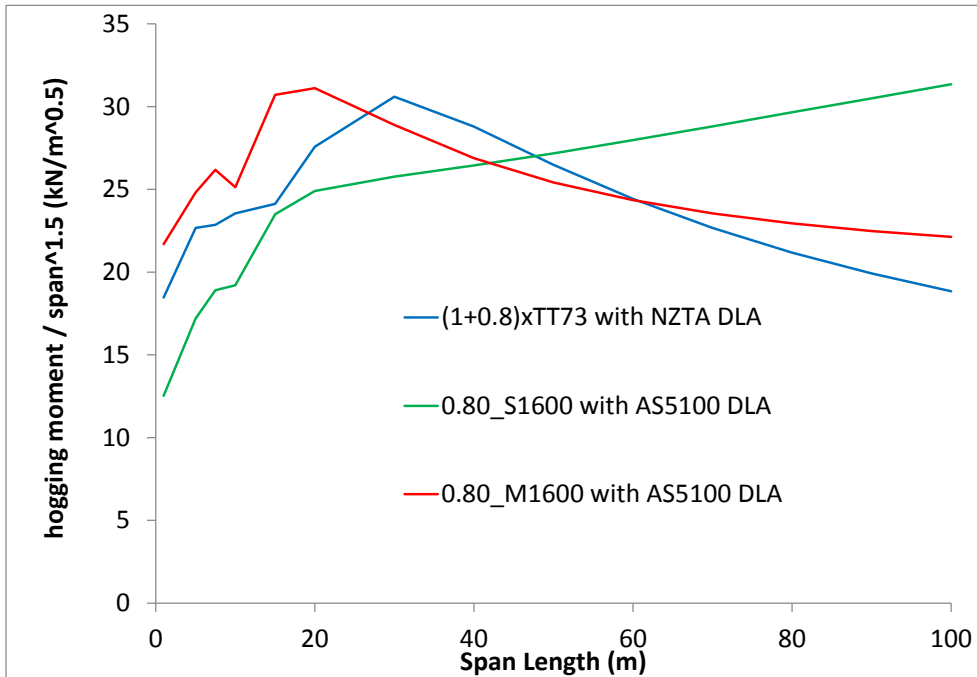
Figure D.29   Modification to the AS5100 loading – shear
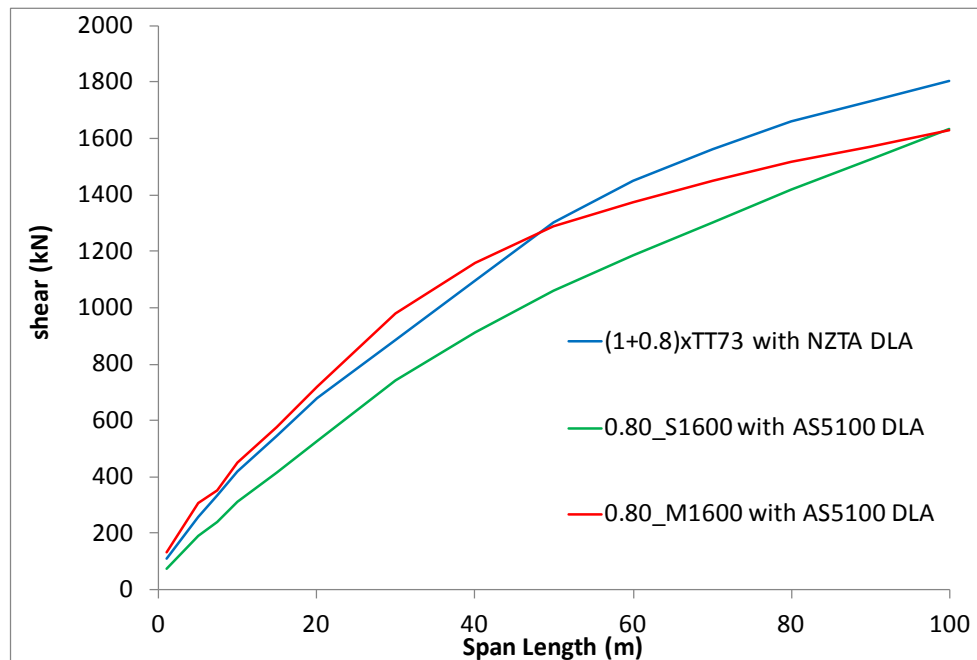
D2.6   Wheel and axle loading from the TT73 vehicle

The TT73 vehicle has nominal axle loads of 8t and 9t on single axles. These axle loads are comparable to current practice, reflecting the fact that the TT73 potential future general access vehicle was created by introducing additional axles rather than by increasing the load per axle. The top down approach does not, therefore, lead to a requirement for increased axle design loads. The design model for axle and wheel loading will be discussed further in section D5.1.

D2.7   Conclusions from the top down approach

The basis of the top down approach to the development of a bridge loading model is to look at the future development of vehicle loads and develop a loading model that will cater for future vehicles. The TT73 vehicle was postulated as representing the potential future general access vehicle. This was based on the truck and full trailer, which is the dominant freight vehicle on New Zealand roads. A freight density of 0.73t per cubic metre was assumed in determining the axle loads from this vehicle.

The loading model was developed on the basis that it would be either a modification to the existing *Bridge manual*, or to the AS5100.

The model was developed by combining long-span loading with short and medium-span loading.

Based upon the top down approach it is recommended that the loading model for a new vehicle loading standard for road bridges in New Zealand should be either:

- 2.0_HN-72 (illustrated in figure D.30), or

- 0.8_SM1600 (illustrated in figure D.31).

The variation of these loading models from the actions due to the TT73 vehicle is summarised in table D.6.

Table D.6   **Ratio of the recommended design model actions to the TT73 actions**

Span	2.0_HN/TT73 (DLA included)			0.8_SM1600/TT73 (DLA included)		
Metres	Sagging moment	Shear	Hogging moment	Sagging moment	Shear	Hogging moment
1	2.85	2.84	1.81	1.22	1.19	1.17
5	1.66	1.48	1.49	1.20	1.20	1.09
7.5	1.51	1.55	1.33	1.19	1.06	1.15
10	1.42	1.44	1.15	1.06	1.08	1.07
15	1.34	1.32	1.10	1.06	1.05	1.27
20	1.23	1.20	**0.97**	1.01	1.06	1.13
30	1.14	1.11	**0.90**	1.08	1.11	**0.94**
40	1.11	1.03	**0.98**	1.16	1.06	**0.93**
50	1.07	**0.98**	1.11	1.14	**0.99**	1.03
60	1.03	**0.98**	1.24	1.09	**0.95**	1.15
70	1.03	1.00	1.38	1.06	**0.93**	1.27
80	1.05	1.02	1.52	1.05	**0.91**	1.40
90	1.08	1.06	1.67	1.05	**0.91**	1.53
100	1.12	1.09	1.82	1.08	**0.91**	1.66

Figure D.30   2.0_HN-72 loading
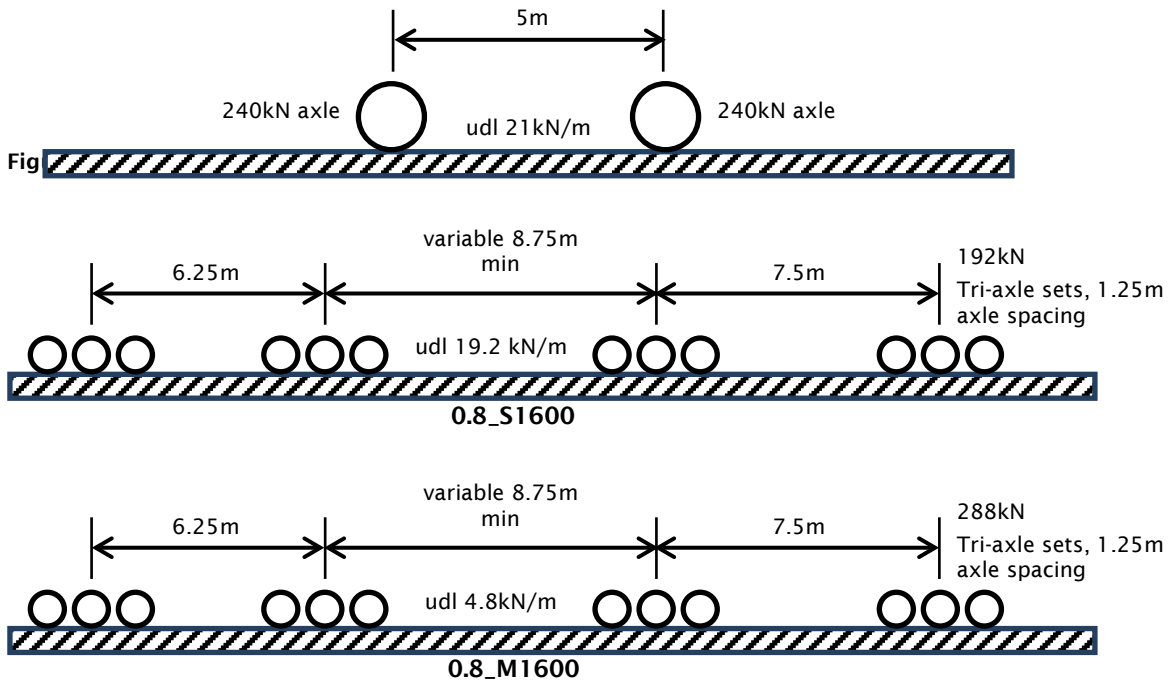
D3   Bottom up approach

The bottom up approach to the development of a bridge loading model is based upon analysing the loading from current traffic, and developing a loading model that will allow for those structural actions.

In this report the loading from current traffic (including vehicles in New Zealand, Australia and Europe) has been approached in several ways:

- Loading due to legally loaded vehicles from New Zealand (Class 1 and HPMV loading), Australia (45.5 tonne semi-trailer and 68t B-double) and Europe (articulated, truck and trailer and longer heavier vehicles) was evaluated.

- The loading from the vehicles proposed in responses to the questionnaire presented in appendix E was evaluated. The questionnaire was distributed to industry via members of the Axle Weights and Loadings Advisory Group (AWLAG).

- Transport Agency permit applications were reviewed in order to determine the loading from vehicles using the road network but exceeding current legal loading.

- The loading due to five, six and eight-axle mobile cranes (commonly referred to as European cranes) was evaluated.

The Transport Agency does not require the bridge loading model to allow for a specific heavy load platform, nor is it a requirement that any specific permit vehicle must be accommodated.

In addition the WiM data presented in appendix C was analysed to determine a loading that was representative of current traffic.

The bending moments and shear forces due to all of these different vehicle loadings were assessed for single and twin continuous spans up to 100m. From these results, design vehicles were developed based on modifications to the *Bridge manual* HN-72 loading and to the AS5100 SM1600 loading.

D3.1   Limit state load factors

The design actions for developing a loading model were calculated by using the nominal loads of the vehicles used in the bottom up approach. As such these can be considered daily events (albeit of large magnitude), rather than events with a 20- or 2000-year return period. Load factors for serviceability or ULS were not considered.

D3.2   Application of a dynamic load allowance

As discussed in section D2.5.1, it is necessary to apply a DLA to the loads from the vehicles used in the bottom up approach to enable comparison with the AS5100 design loading. The DLA applied in the *Bridge manual* has been used for this purpose.

D3.3   Coincident lane loading

When determining the structural actions due to a specific vehicle it is necessary to place additional loading in the same traffic lane to represent other vehicles that are travelling in front of and behind the specific vehicle. This loading will be referred to as the coincident lane loading.

In this research the coincident lane loading was applied to the entire span except for the region beneath the specific vehicle including 5.5m in front of the front axle and 5.5m behind the rear axle (representing headway to leading and following vehicles). The magnitude of the coincident lane loading was based upon data from the WiM analysis. Specifically, the lane loading corresponding to the 95th percentile value of the daily maximum structural action (sagging moment, shear or hogging moment) due to the recorded weight and spacing of traffic over the three year period of the WiM data was used as the coincident lane loading. The 95th percentile value was chosen to represent maximum values from daily traffic, but with the extreme outlier values excluded.

The loading corresponding to the 95th percentile value of the daily maximum structural action was converted to an equivalent udl to place in front of and behind the specific vehicle. The process for determining the value of that udl (which varies with span length) was as follows:

- The daily maximum sagging moment for each of the 905 days of recorded data was plotted in rank order, for each span length, and the 95th percentile value was calculated (figure D.32 is an example for a 40m span).

- The 95th percentile values of maximum sagging moment were plotted against span (figure D.33) and an equation for moment due to a uniform load was fitted to this curve.

- The resulting equation for a udl that gave an equivalent sagging moment to the 95th percentile value of the daily maximum sagging moment was:

$$udl = 140 \times span^{-0.5} \; kN/m$$

- The process was repeated to find a udl that gave an equivalent shear to the 95th percentile value of the daily maximum shear, and the same equation resulted.

- The process was repeated once more to find a udl that gave an equivalent hogging moment to the 95th percentile value of the daily maximum hogging moment for twin continuous spans (plotted in figure D.34) and the resulting equation was:

$$udl = 109 \times span^{-0.54} \; kN/m$$

Figure D.32   Daily moment maxima due to traffic recorded weight and spacing – WiM data 40m single span
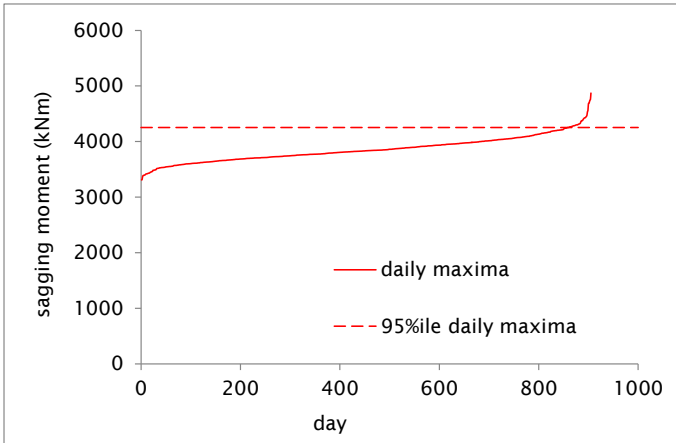
Figure D.33   95th percentile daily sagging moment maxima due to traffic recorded weight and spacing
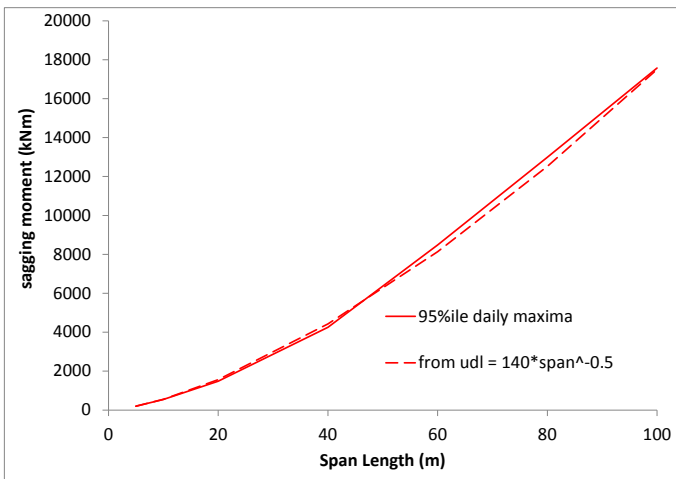
Figure D.34   95th percentile daily hogging moment maxima due to traffic recorded weight and spacing
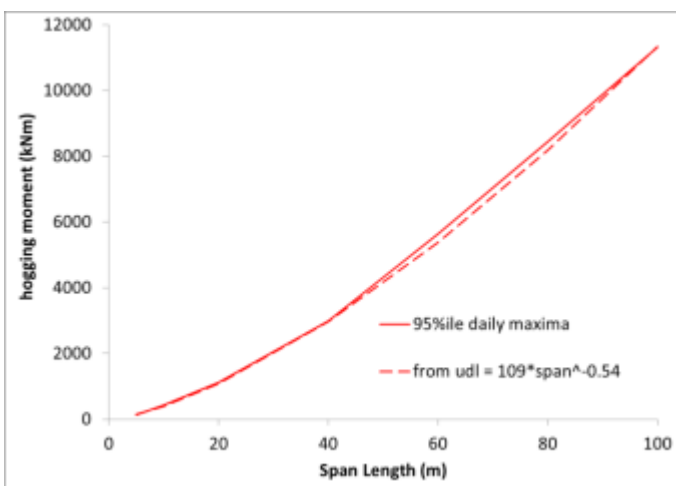
When calculating the structural actions due to a specific vehicle that vehicle was moved across the single or twin span bridge, and the coincident lane loading (the udl in figure D.33 or D.34) was placed on the

bridge except for the region beneath the specific vehicle including 5.5m in front of the front axle and 5.5m behind the rear axle (representing headway to leading and following vehicles).

In appendix C it was noted that there was concern about the validity of the WiM data. Therefore this calculation of the coincident lane loading, which was based on WiM data, was checked using an alternative approach. Section D2.3 described a process for determining the loading on a span based upon the value of the vehicle load on a 30m loaded length. The formula was:

$$\text{load}_x = load_{30}\left[\frac{1}{70} \times loaded\ length + \frac{40}{70}\right]$$

The coincident lane loading was calculated from this formula assuming that the coincident vehicles were a queue of R22T22 vehicles complying with the HPMV rules. The NZ HPMV 2222 vehicle defined in section D3.5 was used for the purpose. This vehicle has a loaded mass of 69t at 30m (based upon two vehicles with 5.5m headway), therefore the coincident lane loading is represented by:

$$\text{load}(\text{tonnes}) = 69\left[\frac{1}{70} \times loaded\ length + \frac{40}{70}\right]$$
$$\approx 1.0 \times loaded\ length + 40$$

That is, a concentrated load of 400kN and a udl of 10kN/m.

The bending moments and shear forces due to a single R22T22 HPMV vehicle with coincident lane loading were calculated using each of the two methods described:

- The coincident lane loading was based on a udl equivalent to the 95th percentile values of maximum structural actions obtained from the WiM data.

- The coincident lane loading was based upon a queue of R22T22 vehicles.

The results are presented in full in annex D1, and the results for bending moment in single spans are plotted in figure D.35. This shows an acceptable agreement between both approaches. Based upon this validation, the coincident lane derived from the WiM analysis was adopted for use.

Figure D.35   Bending moments in single spans for R22T22 HPMV vehicle – alternative coincident lane loading
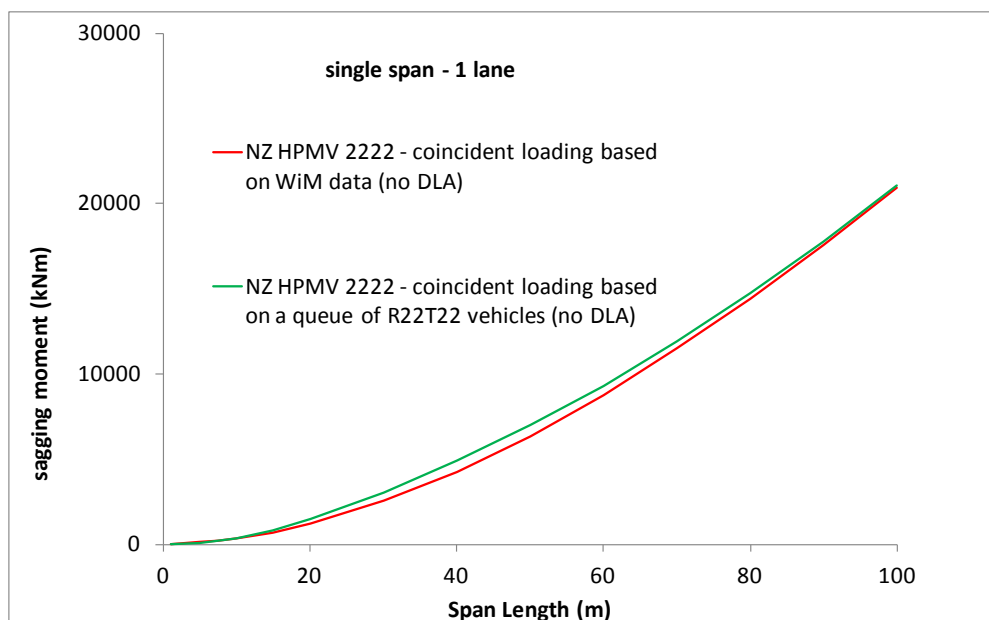
D3.4   Presentation of the analysis results

In undertaking the bottom up approach to the development of the loading model, bending moments and shear forces (structural actions) for simple and twin continuous spans were calculated for spans up to 100m. The calculations were undertaken for the following loadings, as outlined in section D3:

- legally loaded vehicles

- vehicles proposed in the AWLAG questionnaire responses

- Transport Agency permit application vehicles

- five, six and eight axle mobile cranes

- WiM data.

With the exception of the loading from WiM data, the coincident lane loading was applied in each case. A DLA was applied to all of these vehicle loadings, as described in section D3.2.

D3.5   Structural actions from legally loaded vehicles

Structural actions for single and twin continuous spans up to 100m were calculated for eight representative legal vehicles from New Zealand, Australia and Europe. Coincident lane loading was applied in each case.

Detailed axle mass and spacing information is provided for each vehicle in table D.7.

The following vehicles were analysed:

NZ Class 1 2222:

This is a truck and trailer combination that complies with the Class 1 mass and dimension requirements. The vehicle has a gross combination mass of 44t.

NZ HPMV 2222:

This is a truck and trailer combination that complies with HPMV mass and dimension requirements. The vehicle has a gross combination mass of 59t.

Aus 45.5T semi:

This is a semi-trailer that complies with the Australian Higher Mass Limits rules. The gross combination mass is 45.5t.

Aus 68T BD:

This is a B-double vehicle that complies with the Australian Higher Mass Limits rules. The gross combination mass is 68t.

Euro A113:

This is an articulated vehicle that complies with the mass and dimension requirements of the European Union, for travel throughout the member states. The gross combination mass is 42t.

Euro R12T2:

This is a truck and trailer vehicle that complies with the mass and dimension requirements of the European Union, for travel throughout the member states. The gross combination mass is 40t.

Euro LHVa:

This is representative of the longer heavier vehicles (LHVs) that operate in some European countries (Netherlands, Sweden and Finland) and are being trialled in other European countries. The gross

combination mass is 60t. The LHVa vehicle comprises a rigid truck (R12) hauling a trailer (T23) through a converter dolly.

Euro LHVb:

This is representative of the LHVs that operate in some European countries (Netherlands, Sweden and Finland) and are being trialled in other European countries. The gross combination mass is 60t. The LHVb vehicle is a B-train, with a A122 semi-trailer hauling a tri-axle semi-trailer.

Structural actions for single and twin continuous spans up to 100m are plotted in annex D3 for each of these eight vehicles. A representative graph, for bending in simple spans, is shown in figure D.36.

It is apparent from the graphs that the structural actions from all eight vehicles fall within a relatively narrow band. The upper bound to this band of structural actions was used to represent the loading from legal vehicles.

The upper bound structural actions from legal vehicles should be regarded as a lower bound to the bottom up loading model.

Table D.7   Configuration of legal vehicles

Designation	Overall axle spacing (m)	GCM (tonnes)	Axle mass (tonnes) and spacing (metres)								
			1	2	3	4	5	6	7	8	9
NZ Class 1 2222	16	44	5	5	5	5	6	6	6	6	
				1.3	3.7	1.3	3.4	1.3	3.7	1.3	
NZ HPMV 2222	20.5	59	5.5	5.5	8	8	8	8	8	8	
				1.3	4.2	1.3	5.55	1.3	5.55	1.3	
Aus 45.5T semi	11	45.5	6	8.5	8.5	7.5	7.5	7.5			
				3	1.2	4.4	1.2	1.2			
Aus 68T BD	21	68	6	8.5	8.5	7.5	7.5	7.5	7.5	7.5	7.5
				3	1.2	6.5	1.2	1.2	5.5	1.2	1.2
Euro A113	11.54	42	6.7	11.3	8	8	8				
				3.6	5.32	1.31	1.31				
Euro R12T2	14.31	40	7	8	8	8.5	8.5				
				4.8	1.4	6.31	1.8				
Euro LHVa	22.252	60	8	7.75	7.75	7.75	7.75	7	7	7	
				4.5	1.37	5.015	1.31	7.337	1.36	1.36	
Euro LHVb	26.207	60	7	8	8	8	8	7	7	7	
				3	1.37	10.21	1.81	7.097	1.36	1.36	

Figure D.36   Bending moments in simple spans due to loading from representative legal vehicles
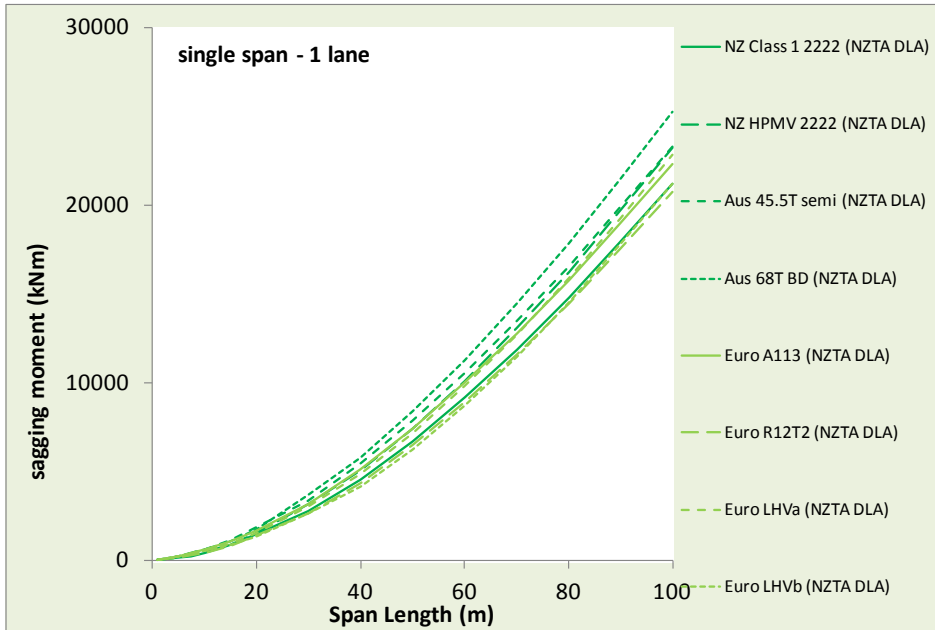
D3.6   Structural actions for vehicles proposed in the AWLAG questionnaire responses

A questionnaire distributed to industry via members of the AWLAG as part of this research project is included in appendix E. One of the questions asked 'what vehicles would you like to be able to use on the public highways in the future?' Eight vehicle configurations were received in responses to the questionnaire.

The axle mass and spacing configurations of those eight vehicles are listed in table D.8.

Structural actions for single and twin continuous spans up to 100m are plotted in annex D4. Coincident lane loading as described in section D3.3 was applied in each case. A representative graph, for bending in simple spans, is shown in figure D.37.

One vehicle, with 11 axles and a mass of 145t, dominated the structural actions. Because of its large weight the researchers considered it inappropriate to include this vehicle as an input into determining the bottom up loading model, which is a representation of current traffic. Therefore, the envelope of maximum structural actions due to the AWLAG response vehicles was calculated excluding this vehicle.

Table D.8   Vehicle configurations submitted with the AWLAG questionnaire responses

Designation	GCM (tonnes)	Axle mass (tonnes) and spacing (metres)										
		1	2	3	4	5	6	7	8	9	10	11
Heb Structures	111	7	9	9	13	13	15	15	15	15		
			4.318	1.375	3.03	1.24	6.5	2.4	2.4	2.4		
Waikato Crane Services	145.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2	13.2
			1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65	1.65
Unidentified 1	69.5	6.5	9	9	15	15	15					
			4.5	1.4	6.5	2.4	2.4					
Emmerson Transport	52	5	5	7.5	7.5	6.75	6.75	6.75	6.75			
			1.88	4	1.32	4.8	1.2	6.18	1.32			

Designation	GCM (tonnes)	Axle mass (tonnes) and spacing (metres)										
		1	2	3	4	5	6	7	8	9	10	11
TNL Group	63	6	7.5	7.5	7	7	7	7	7	7		
			4	1.3	3.4	1.5	1.5	6	1.5	1.5		
McIntosh Brothers	56.6	11.35	11	11.5	11.35	11.4						
			2.54	1.65	2	1.65						
L&A Cotton	108.6	9	9	9	12.5	12.5	14.15	14.15	14.15	14.15		
			4.25	1.33	2.92	1.3	6.47	2.4	2.4	2.4		
NZ Express Transport	58.8	5.4	5.4	8	8	8	8	8	8			
			1.3	2.5	1.3	7	1.3	1.3	1.3			

Figure D.37   Bending moments in simple spans due to loading from AWLAG questionnaire vehicles
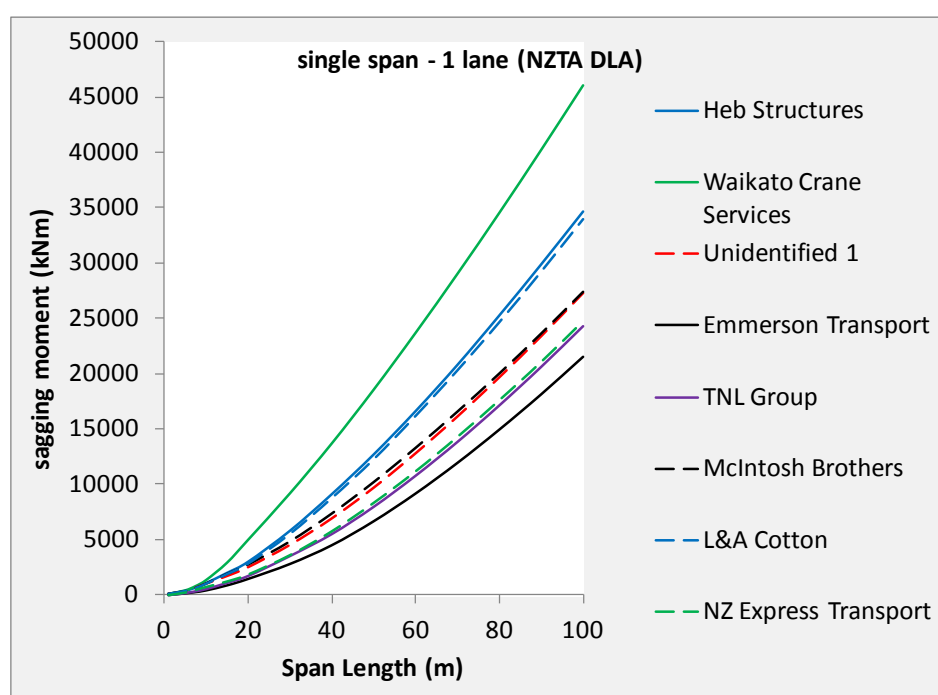
D3.7   Structural actions from Transport Agency permit vehicles

The structural actions were calculated for each of the vehicles in 17,743 permit applications provided to the researchers by the Transport Agency. The permit applications were divided into 14 vehicle categories.

Table D.9 lists the vehicle categories, the number of vehicles in each category, and the maximum structural actions for each category over a range of single and twin continuous spans up to 100m.

For medium to long spans the structural actions were dominated by two vehicle categories: 'large project transporter' and 'transporter'. This is consistent with the category descriptions, which suggest that these are exceptionally heavy loads. Because of this the researchers considered it inappropriate to include these permit vehicle categories as an input into determining the bottom up loading model, which is a representation of current traffic.

For spans of 10m and below the structural actions were dominated by 'dump trucks'. Checking of the permit vehicle data shows that these are two and three axle trucks with short wheel base and axle loads of

up to 35t. Again, because of these exceptional axle loads, the researchers considered it inappropriate to include this permit vehicle category as an input into determining the bottom up loading model.

Therefore the envelope of maximum structural actions due to the permit vehicles was calculated excluding the categories 'large project transporter', 'transporter' and 'dump truck'. Excluding these vehicle, 5143 permit vehicles remained. From these 5143 permit vehicles, the category that caused the maximum structural action over the full range of single and twin continuous spans up to 100m was 'mobile crane' in nearly all cases.

Table D.9   Permit application vehicles – vehicle summary and structural actions

Span	Number:	Agricultural Vehicle	Building Removal Vehicle	Dump Truck	HPMV	ISO Container Truck	Large Project Transporter	Limited Trial Proposal	Mobile Crane	Mobile Plant	Motor Scraper	Mobile X-Ray Vehicle	Slurry Truck	Tow Truck	Transporter	Maximums	Maximums excl. transporters
Span	Number:	524	75	91	20	2036	55	29	1949	407	53	16	16	18	12454	17743	5234
100m	Msag (kNm)	14893	13982	15645	12706	14620	153755	9960	22870	21134	8246	5598	11100	9775	137168	153755	22870
	Vsingle (kN)	615	580	640	522	593	6161	415	942	876	343	229	448	404	5506	6161	942
	Mhog (kNm)	6231	6042	6298	5411	6214	70476	4185	9721	9155	3400	2248	4435	4104	63364	70476	9721
	Vtwin (kN)	632	606	648	543	619	6634	426	978	915	349	231	454	416	5946	6634	978
80m	Msag (kNm)	11651	10794	12363	9841	11338	113381	7777	17721	16258	6473	4449	8791	7620	100368	113381	17721
	Vsingle (kN)	604	565	636	510	576	5761	408	919	851	340	227	444	397	5117	5761	919
	Mhog (kNm)	4946	4743	5030	4284	4914	53242	3325	7677	7224	2710	1795	3543	3262	49613	53242	7677
	Vtwin (kN)	625	593	645	534	607	6271	421	962	897	347	231	452	411	5585	6271	962
60m	Msag (kNm)	8370	7620	9082	6999	8042	74023	5576	12541	11565	4712	3286	6475	5466	65047	74023	12541
	Vsingle (kN)	587	541	630	489	548	5095	397	881	810	335	225	438	386	4470	5095	881
	Mhog (kNm)	3651	3460	3760	3139	3593	36923	2456	5598	5255	2016	1340	2648	2414	33848	36923	5598
	Vtwin (kN)	613	573	641	518	585	5623	413	932	864	343	229	449	403	4969	5623	932
40m	Msag (kNm)	5088	4644	5801	4134	4757	39278	3373	7422	7004	2933	2104	4168	3374	32177	39278	7422
	Vsingle (kN)	552	492	617	446	506	4096	376	804	754	324	220	426	363	3362	4096	804
	Mhog (kNm)	2324	2130	2482	1957	2224	21857	1568	3448	3206	1313	883	1748	1548	19911	21857	3448
	Vtwin (kN)	585	531	633	482	536	4549	396	865	801	336	226	441	385	3815	4549	865
20m	Msag (kNm)	1861	1799	2519	1458	1734	10465	1353	2816	2741	1266	942	1857	1493	8044	10465	2816
	Vsingle (kN)	448	382	577	335	422	2281	311	617	597	293	207	390	334	1754	2281	617
	Mhog (kNm)	1043	1160	1178	882	1038	9354	690	1722	1611	575	415	831	675	8082	9354	1722
	Vtwin (kN)	485	417	603	365	454	2714	334	679	646	308	215	413	348	2153	2714	679
10m	Msag (kNm)	710	565	980	474	712	2710	500	1001	1025	497	381	734	590	2083	2710	1025
	Vsingle (kN)	313	271	497	222	298	1235	241	460	440	230	179	317	291	949	1235	497
	Mhog (kNm)	510	409	536	382	496	2668	368	710	614	341	213	393	369	2051	2668	710
	Vtwin (kN)	351	302	528	247	320	1489	261	503	493	243	188	345	306	1145	1489	528
5m	Msag (kNm)	204	200	435	160	269	722	198	372	270	249	161	294	265	555	722	435
	Vsingle (kN)	209	179	362	153	226	728	175	310	254	195	149	257	231	560	728	362
	Mhog (kNm)	161	153	309	128	145	671	148	260	247	113	92	184	156	516	671	309
	Vtwin (kN)	234	196	391	168	248	853	191	345	294	197	157	275	240	656	853	391

Msag = maximum sag moment in simply supported span

Vsingle = maximum shear in simply supported span

Mhog = maximum hog moment in twin continuous span

Vtwin = maximum shear in twin continuous span

D3.8   Structural actions from European cranes

The loading due to a selection of mobile cranes (commonly referred to as European cranes) was calculated for consideration in the bottom up approach to the loading model. Five, six and eight-axle cranes were considered.

The axle mass and spacing configurations of those cranes are listed in table D.10.

Table D.10   Vehicle configurations for European cranes

Designation	GCM (tonnes)	Axle mass (tonnes) and spacing (metres)							
		1	2	3	4	5	6	7	8
5-axle crane	60	12	12	12	12	12			
			2.5	1.7	2	1.7			
6-axle crane	72	12	12	12	12	12	12		
			1.9	2.9	1.7	2.6	1.7		
8-axle crane	85.5	12	12	12	12	12	8.5	8.5	8.5
			1.5	1.5	2	1.5	2.4	1.5	2.5

D3.9 Envelope of structural actions from AWLAG, permits and cranes

The vehicles proposed in the AWLAG questionnaire responses, the vehicles in the Transport Agency permit applications, and the European cranes are all heavy vehicles. The structural actions from all of these vehicles were calculated assuming that the DLA varied with span, from 0.3 at 1m span to 0.11 at 100m span (as per the *Bridge manual*). If these vehicles moved across bridges in a controlled manner at low speeds, it may be possible to reduce this DLA; however, such control cannot be assured for these vehicles, and has not been assumed in the development of the loading model.

It is, however, common to adopt a reduced load factor when calculating the ULS loading from vehicles such as these on the basis that the loads are more accurately known and better controlled. Therefore, for the purpose of comparing the loading from these vehicles to the other vehicles considered in the bottom up approach, a factor of 0.8 was applied to the structural actions from the vehicles proposed in the AWLAG questionnaire responses, the vehicles in the Transport Agency permit applications and the European cranes.

Structural actions for single and twin continuous spans up to 100m are plotted in annex D5 for:

* the envelope of maximum structural actions from the AWLAG questionnaire response vehicles

* the envelope of maximum structural actions from the Transport Agency permit applications

* the structural actions from the three European cranes.

Coincident lane loading as described in section D3.3 is included in all cases.

A representative graph, for bending in simple spans, is shown in figure D.38.

Figure D.38   Bending moments in simple spans due to AWLAG vehicles, permit vehicles and cranes
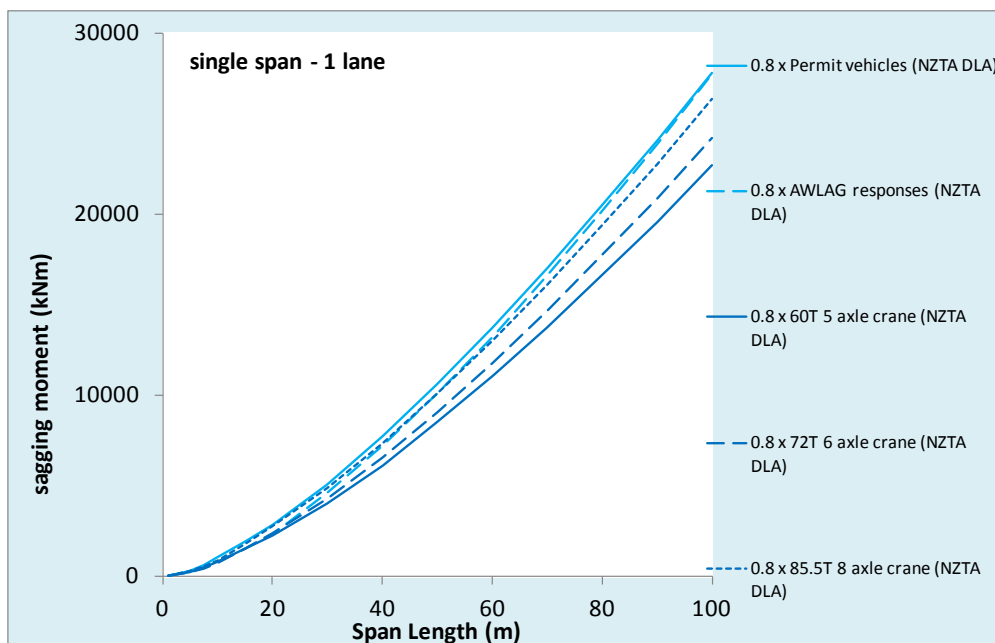
The envelope of the maximum structural actions from AWLAG questionnaire responses, the vehicles in the Transport Agency permit applications, and the European cranes should be regarded as a desirable, but not necessary, upper bound to the bottom up loading model.

D3.10 Structural actions from the WiM analysis

The 20-year return period WiM analysis results in appendix C were used to represent design actions for consideration in the bottom up approach. It is acknowledged that there is an inconsistency in comparing 20-year return period loading data with the nominal loads from legally loaded vehicles, AWLAG response vehicles, permit vehicles and mobile cranes. These specific vehicles can be considered as daily events (albeit of large magnitude). This inconsistency goes to the definition of the SLS loading, and whether this is a true probabilistic loading (based on a 20-year return period), or is in reality better defined as the maximum daily loading event. This is not treated consistently in current design codes, and resolution of this issue is outside the scope of this research report.

Concern about the validity of the WiM data was expressed in appendix C, section C2.5, and it was noted that the analysis of the WiM data, and the conclusions from that analysis, may therefore be unreliable. The WiM data is presented here with that caveat also. However it will become apparent in the following sections that the WiM data was not a significant factor in determining the recommended loading model.

Structural actions for single and twin continuous spans up to 100m are plotted in annex D2 for the three WiM loading scenarios: recorded weight and spacing of traffic; 1% queued; and 4% queued (refer to appendix C for additional explanation).

A representative graph, for bending in simple spans, is shown in figure D.39.

It is apparent from the graphs that the structural actions from queued traffic are not sensitive to the assumption about the frequency of queue formation – the results for 1% and 4% queue formation are very similar.

From each graph the maximum structural actions were extracted, whether from moving traffic (WiM recorded weight and spacing of traffic) or from stationary traffic based upon 4% queue formation.

Figure D.39 **Bending moments in simple spans due to loading from WiM data analysis**
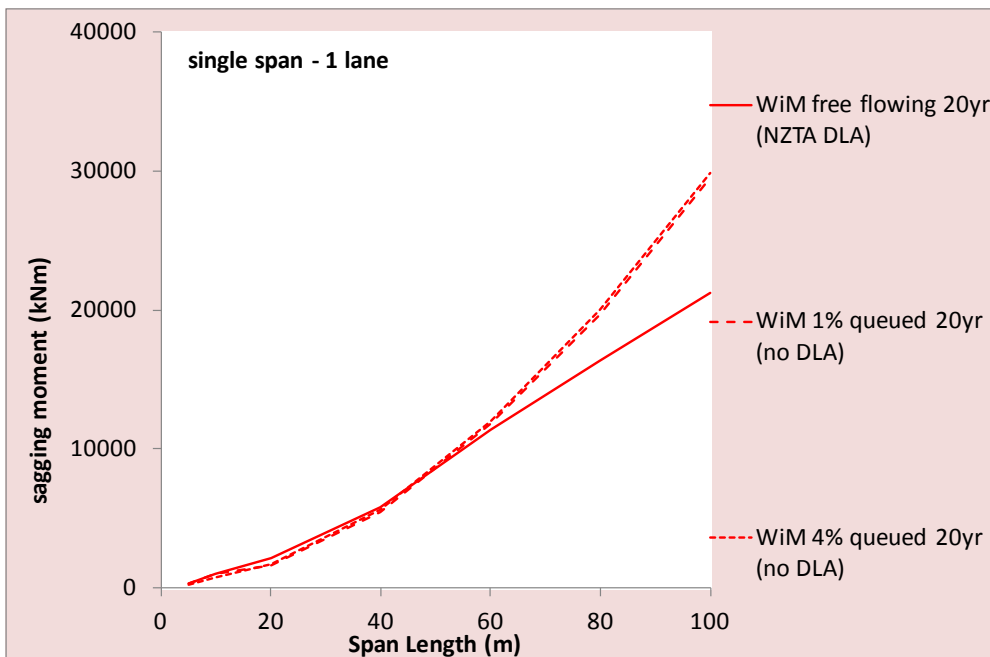
D3.11 Combining the inputs into the bottom up loading model

Development of the bottom up loading model has considered the structural actions from the following loadings, as outlined in section D3:

- legally loaded vehicles

- vehicles proposed in the AWLAG questionnaire responses

- Transport Agency permit application vehicles

- five, six and eight-axle mobile cranes

- WiM data.

With the exception of the loading from WiM data, the coincident lane loading as described in section D3.3 was applied in every case.

The envelopes of maximum structural actions from each of these five loading types are plotted in annex D6, and a representative graph, for bending in simple spans, is shown in figure D.40.

Figure D.40   Upper bound of bending moments in simple spans due to WiM, legal, AWLAG permit and cranes
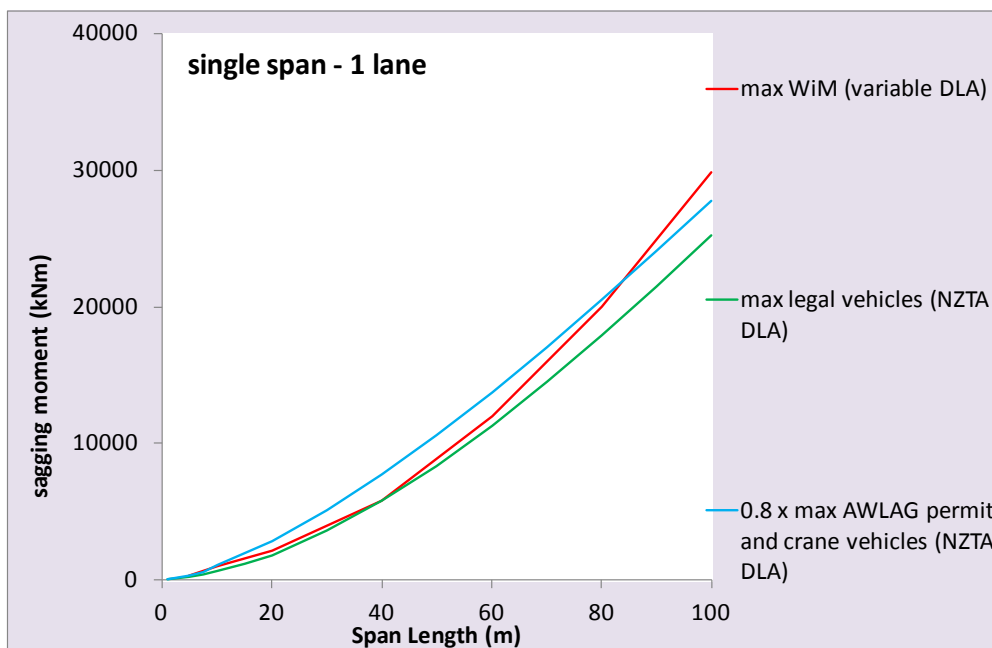
D3.11.1   Modification to the *Bridge manual* loading model

The HN-72 loading model was factored to fit to the upper bound of the actions from the vehicles considered in the bottom up approach: legally loaded vehicles; the vehicles proposed in the AWLAG questionnaire responses; the vehicles taken from the Transport Agency permit applications; five, six and eight-axle mobile cranes; and the WiM data with a 20-year return period.

The upper bound actions from the bottom up approach together with the structural actions from the factored HN-72 loading model are plotted for single and twin continuous spans up to 100m in annex D7, and a representative graph, for bending in simple spans, is shown in figure D.41. The variation of the loading model from the upper bound structural actions is summarised in table D.11.

Figure D.41   Factored HN-72 loading model compared to upper bound sagging bending moment
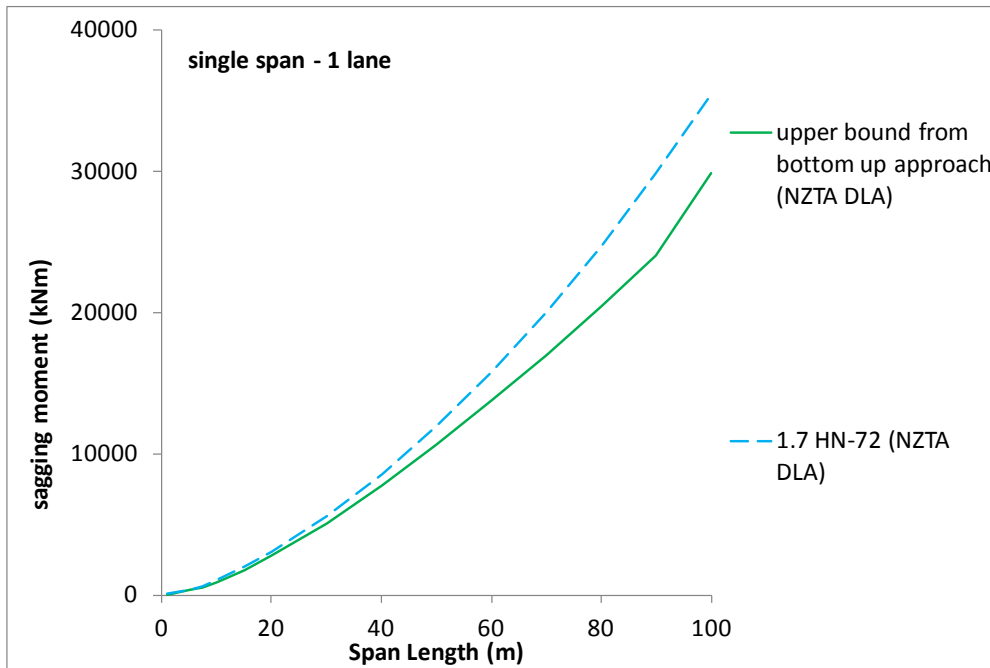
Table D.11   Factored HN-72 loading model compared with the upper bound of bottom up vehicle actions

Factored HN-72 loading model			
Span	1.7_HN-72/(upper bound of bottom up vehicle actions)		
Metres	Sagging moment	Shear	Hogging moment
5	1.23	1.09	1.25
7.5	1.23	1.16	1.15
10	1.11	1.11	**0.98**
15	1.12	1.13	**0.97**
20	1.08	1.12	1.07
30	1.09	1.11	1.00
40	1.10	1.13	1.04
50	1.12	1.17	**0.97**
60	1.14	1.22	**0.98**
70	1.17	1.26	**0.96**
80	1.21	1.32	**0.97**
90	1.24	1.38	**0.95**
100	1.19	1.34	**0.96**

D3.11.2   Modification to the AS5100 SM1600 loading model

The AS5100 SM1600 loading model was factored to fit to the upper bound of the actions from the vehicles considered in the bottom up approach: legally loaded vehicles; the vehicles proposed in the AWLAG

questionnaire responses; the vehicles taken from the Transport Agency permit applications; five, six and eight-axle mobile cranes; and the WiM data with a 20-year return period.

The upper bound actions from the bottom up approach together with the structural actions from the factored AS5100 loading model are plotted for single and twin continuous spans up to 100m in annex D7, and a representative graph, for bending in simple spans, is shown in figure D.42. The variation of these loading models from the upper bound structural actions is summarised in table D.12.

Figure D.42   Factored SM1600 loading model compared to upper bound sagging bending moment
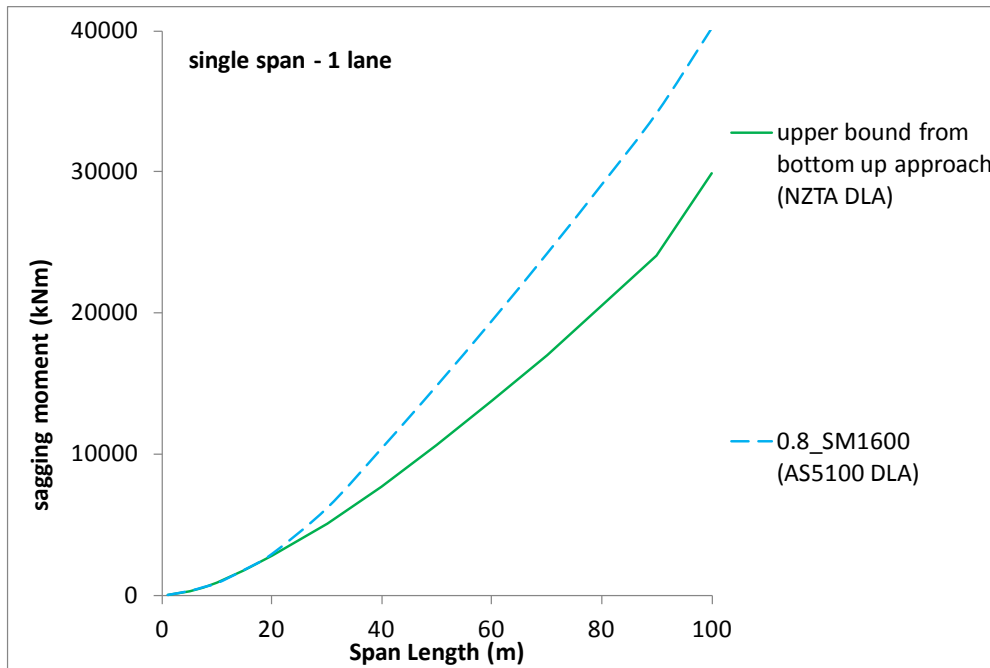
Table D.12   Factored SM1600 loading model compared with upper bound actions

Factored SM1600 loading model		
Span	0.8_SM1600/(upper bound of bottom up vehicle actions)	
Metres	Sagging moment	Shear	Hogging moment
5	1.05	1.03	1.08
7.5	1.14	**0.93**	1.16
10	**0.97**	**0.98**	1.08
15	1.04	1.05	1.32
20	1.05	1.15	1.46
30	1.22	1.31	1.24
40	1.35	1.36	1.16
50	1.40	1.39	1.06
60	1.42	1.40	1.07
70	1.42	1.39	1.04
80	1.42	1.39	1.05
90	1.42	1.39	1.03
100	1.35	1.31	1.03

D3.12 Conclusions from the bottom up approach

Analysis of the structural actions from the following loadings:

- legally loaded vehicles

- vehicles proposed in the AWLAG questionnaire responses

- Transport Agency permit application vehicles

- five, six and eight-axle mobile cranes

- WiM data.

has resulted in the following recommended loading models for a new vehicle loading standard for road bridges in New Zealand:

| Bottom up | short/medium-span loading modification to the *Bridge manual* loading | 1.7_HN |
| Bottom up | short/medium-span loading modification to the AS5100 loading | 0.80_SM1600 |

D4   Recommended loading model

The researchers formed the view that a new loading standard for New Zealand should be achieved by modifying either the existing *Bridge manual*, or modifying the AS5100. Consistent with that approach, each stage of the development of a new loading model has looked first at possible modifications to the *Bridge manual*, and second at how the AS5100 could be modified for the purpose.

The development of a new vehicle loading model has combined a top down and a bottom up approach. The top down approach considered the future freight need and likely configuration of vehicles to meet that need, and the bottom up approach was based on analysis of the WiM data and effects of current traffic.

The top down and bottom up loading models must be combined to arrive at a final recommendation for a new vehicle loading standard for road bridges in New Zealand. The recommendations from the two approaches are summarised in table D.13.

Table D.13   Top down and bottom up recommended loading models

	Based on modification of:	
	***Bridge manual* loading**	**AS5100 SM1600 loading**
Top down	2.0_HN-72	0.8_SM1600
Bottom up	1.7_HN-72	0.8_SM1600

The recommended bottom up model equals (for the AS5100 loading) or approximates (for the *Bridge manual* loading) the recommended top down model. This indicates that the upper bound of actions from the AWLAG vehicles, permit vehicles and European cranes (with a 0.8 factor) and WiM data are approximately the same as the actions from the TT73 future vehicle.

Combining the top down and bottom up approaches leads to the following recommended loading models:

| Modification to the *Bridge manual* loading | 2.0_HN |
| Modification to the AS5100 loading | 0.80_SM1600 |

The actions from the recommended loading models are compared with the structural actions from the *Bridge manual* HN-HO loading (with SLS factor of 1.35 included) and 0.85HN-72 nominal loading for single and twin continuous spans up to 100m in annex D8, and a representative graph, for bending in simple spans, is shown in figure D.43.

The actions from the recommended loading models are a maximum of 50% greater than the *Bridge manual* HN-HO loading (with SLS factor of 1.35 included).

The recommended loading models are tabulated in table D.14, with the loading diagrams shown in figure D.43

The recommended loading models do not include a separate vehicle to represent overload loading (such as the HO vehicle in the current *Bridge manual*). The Transport Agency does not require the loading model to allow for a specific heavy load platform, nor is it a requirement that any specific permit vehicle must be accommodated. However a range of heavy vehicles were considered and allowed for in developing the loading model, as described in the bottom up approach. It is for these reasons that the researchers consider that a separate vehicle to represent overload loading is not necessary.

Figure D.43   Comparison between the recommended loading models and HN-HO loading
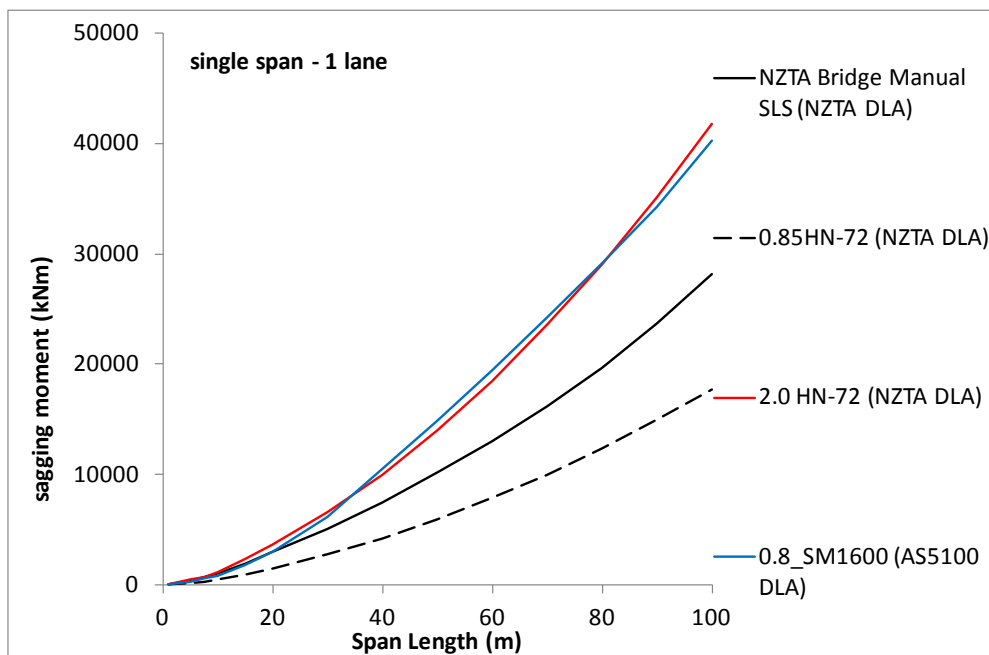
Table D.14   Axle and udl loads for the recommended loading models*

Based on modifications of:		
Bridge manual HN-72 loading	AS5100 SM1600 loading	
2.0_HN-72	0.8_SM1600	
	0.8_M1600	0.8_S1600
axle load: 240kN udl: 21kN/m	axle load: 96kN tri-axle load: 288kN udl: 4.8kN/m	axle load: 64kN tri-axle load: 192kN udl: 19.2kN/m

*Note – Recommendations for individual axle and wheel loads are provided in section D5.1.

Figure D.44 *Bridge manual* **HN-72 and AS5100 SM1600 loading diagrams**
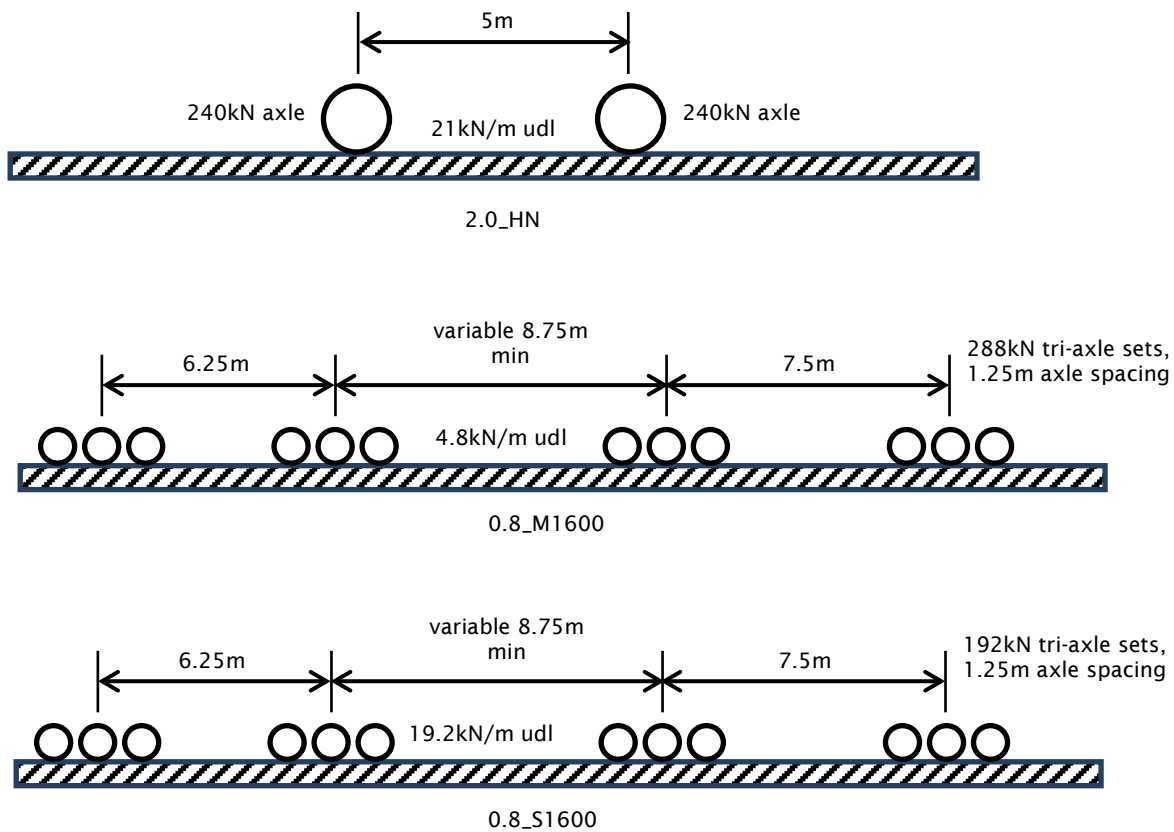
The researchers believe that either of the recommended loading models (based on the *Bridge manual* loading or the AS5100 SM1600 loading) could be implemented in a new vehicle loading standard for the design and evaluation of road bridges in New Zealand. There are, however, some inherent properties of the AS5100 SM1600 model that make this a more appealing loading model.

Adoption of a loading model which is a simple factor of the AS5100 SM1600 loading model facilitates the high level objective of harmonising standards between New Zealand and Australia. Furthermore, the Australian bridge loading is incorporated into many leading bridge design computer programs, including international software provided by leading vendors. Adoption of a loading model which is a simple factor of the AS5100 SM1600 loading model will make this software available for bridge design in New Zealand.

As vehicle loads increase the trend in vehicle design is to increase the number of axles, rather than the load per axle. The SM1600 loading model, with its multi-axle configuration, provides a more realistic representation of the magnitude and distribution of axle loads on bridges. The HN-72 loading model incorporates only two axles, so in order to represent the multi axle combination vehicles that are becoming the predominant freight vehicle, it is necessary to increase the axle loads in the HN-72 model to unrealistically large values. This is evident in the recommended loading models in Table D4.2, where the axle loads for the SM1600 model are 96kN (9.8t) and the axle loads for the HN-72 model are 240kN (24.5t). The latter are unrealistically large, and will cause anomalies in the design for local effects (for example in deck slab design). The researchers believe that design is better served by having a logically consistent set of loads which can be applied to the design of all bridge elements.

For these reasons the recommendation is to adopt 0.8_SM1600 as the loading model.

D5   Other vehicle loads and load factors

Sections D1 to D4 of this appendix have covered the development of a new vehicle loading model for vertical traffic loading. The recommendation was to adopt a factored AS5100 SM1600 loading model. An alternative based on retaining the HN-72 loading model was also presented.

In this section the other aspects of a new vehicle loading standard are discussed, and recommendations are presented. This section considers the following aspects:

*   axle and wheel loading

*   lane widths and number of lanes

*   multiple presence

*   dynamic load allowance

*   horizontal loads

*   load factors.

The approach taken to these aspects of vehicle loading in bridge design standards from New Zealand, Australia, USA, Canada, Europe and the UK was discussed in the literature review in appendix A, and this will be referred to in the following sections.

D5.1   Axle and wheel loading

Figures A.29 and A.30 from the literature review in appendix A are reproduced as figures D.45 and D.46.

Figure D.45   Axle loads
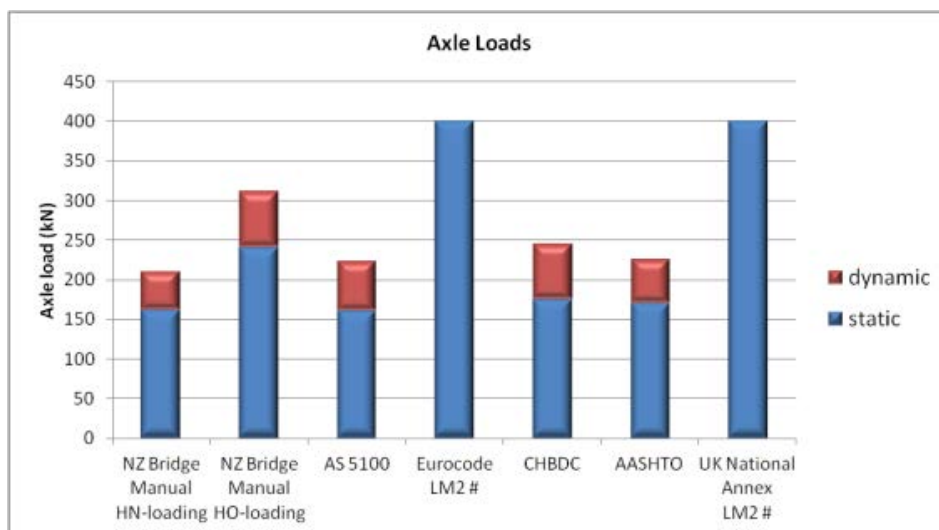
Dynamic effects already included in loading

Figure D.46   Wheel load pressure
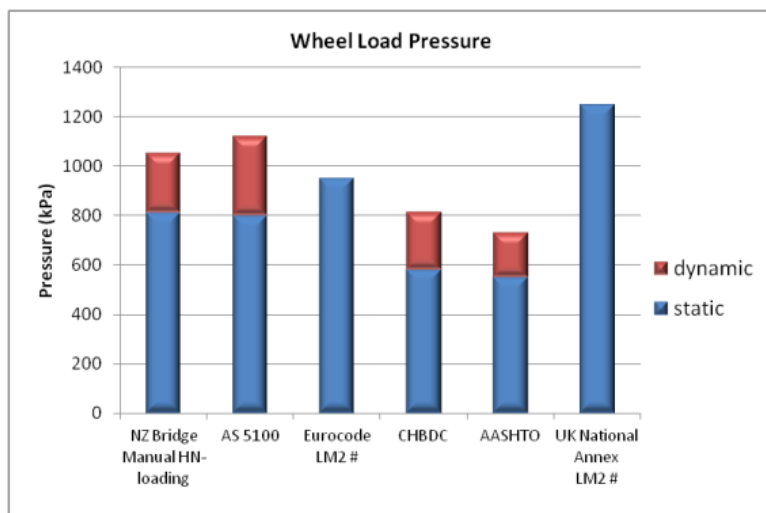
Dynamic effects already included in loading

The *Bridge manual* HO-72 axle loading of 240 kN plus a DLA of 0.3 is the second largest among the bridge standards that were reviewed (second only to the European standards where the legal axle loads can be much greater than in New Zealand). In section D4 it was recommended that the loading model should be either 2.0_HN-72 or 0.8_SM1600.

If 2.0_ HN-72 is adopted, the HN-72 axle load will be 240kN, plus a DLA of 0.3. It is recommended that this axle load be adopted, and that the requirement for HO-72 axle loading is deleted.

If 0.8_SM1600 is adopted as the new vehicle loading model, the AS5100 axle loading should be adopted for consistency. The A160 axle loading in the AS5100 is a 160kN axle loading (with DLA of 0.4). The axle loading should not be reduced by the 0.8 factor. The A160 axle loading should apply.

Referring to figure D.46, the wheel load pressure of 1050 kPa (including DLA) is derived from the HN-72 axle loading and contact area. The wheel load pressure is comparable to that in the AS5100. If 2.0_ HN-72 is adopted as the new vehicle loading model the wheel load pressure will rise accordingly, even though the increased design load is not based on a corresponding increase in legal axle loads. It will therefore be necessary to introduce a design single axle, with a load less than the axle load in the design vehicle, to be used for local actions. This is undesirable, and is an anomaly caused by the HN-72 loading model having only two axles (as discussed in section D4).

If 0.8_SM1600 is adopted as the new vehicle loading model the wheel load pressure should be based upon the AS5100 A160 axle loading and contact area, without the 0.8 factor.

D5.2   Lane widths and number of lanes

The literature review observed that each region has a different methodology to determine lane widths and the number of lanes to be used for design.

The width of one standard traffic lane in New Zealand and in Australia is 3.5m. The bridge design lane width in the *Bridge manual* is 3.0m and in the AS5100 it is 3.2m.

When determining the number of lanes to be used for bridge design, there is a common trend of the design codes where the following occurs:

Design lane width ≤ step increment width for an additional lane ≤ standard traffic lane width

As discussed in the literature review, the *Bridge manual* does not conform to this approach. The step increment for an additional lane is 3.7m – well in excess of the design lane width. The result of this is that the average lane width increases as the bridge width increases. This is illustrated in figure D.47, where the lower bound average design lane width is plotted against the bridge deck width. The *Bridge manual* goes from having the second smallest average lane width for two lanes, to the second largest average lane width for six lanes.

Figure D.47   Lower bound of average design lane width as a function of bridge width
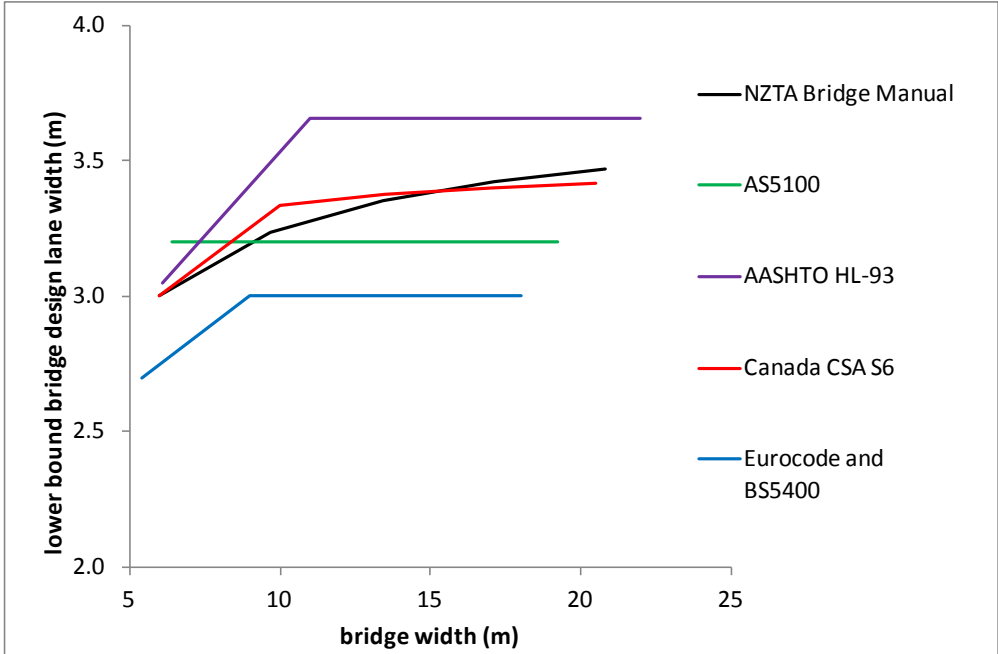
The researchers recognise that the mountainous terrain is a valid reason for adopting a narrower design lane width for two-lane bridges in New Zealand than the value in the Australian bridge design code. However there is no apparent reason for the step increment of 3.7m for additional lanes.

It is recommended that the current requirements for the number of bridge design lanes be replaced by the requirements of the AS5100, that the number of design lanes is equal to the bridge width divided by 3.2 (rounded down to the next integer), with the additional requirement that bridges with a width between 6 and 6.4m are designed for two lanes. This recommendation is shown in table D.15 together with the current requirements of the other bridge design codes.

Table D.15   Maximum bridge width as a function of the number of bridge design lanes

Number of bridge design lanes	Maximum bridge width for the number of lanes (m)					
	Bridge manual	*recommended*	**AS5100**	**AASHTO HL-93**	**Canada CSA S6**	**Eurocode and BS 5400**
1	6	*6*	6.4	6.1	6	5.4
2	9.7	*9.6*	9.6	11.0	10	6
3	13.4	*12.8*	12.8	14.6	13.5	9
4	17.1	*16*	16	18.3	17	12
5	20.8	*19.2*	19.2	21.9	20.5	15
6	24.5	*22.4*	22.4	25.6	24	18

D5.3   Multiple presence

The literature review noted that the trend of dealing with the multiple presence of vehicles in adjacent lanes by applying lane reduction factors to additional individual lanes (rather than applying a load reduction factor to all loaded lanes concurrently) is consistent with the probabilistic design philosophy of load factors and load combinations, where the critical loading is combined with an additional transient effect. The additional transient effect is reduced to account for the reduced probability of concurrent loading.

The *Bridge manual* is not in line with this trend, and applies a load reduction factor to all loaded lanes concurrently, effectively averaging the traffic loading across all lanes. This is a particular issue when a heavy vehicle in one lane is combined with traffic in other lanes, and the load from the heavy vehicle becomes 'averaged out'.

It is recommended that the traffic loading model be revised to incorporate lane reduction factors for additional individual lanes, and that the accompanying lane factors in the AS5100 be adopted.

D5.4   Dynamic load allowance

Studies on DLAs are reviewed in appendix A and summarised as follows:

- The dynamic increase is dependent on a number of factors including but not limited to:
    - type of vehicle, mass and speed
    - the road profile
    - the span length
    - the frequency and damping of the bridge
    - the number of axles loaded on the bridge
    - the number of lanes loaded on the bridge.

- There is a common trend for the dynamic increase to reduce towards zero for longer-span bridges. This is achieved through the design codes in two ways. One is through an equation, where the factor reduces with increased length. The other is where the uniformly distributed loading component, which has no dynamic factor applied, becomes the dominant component for longer-span bridges.

- For medium-span bridges, the DLA is typically in the range of 0.25 to 0.35.

- The main trend of the codes, however, is to apply a simplified procedure for determining and applying the DLAs.

The *Bridge manual* retains the equation for DLA that was introduced in the 1931 first edition of the AASHO *Standard specification for highway bridges*, but was superseded by AASHTO after the 17th edition in 2002.

Referring again to the recommendations in section D.4 that to allow for future freight needs the loading model should be either 2.0_HN-72 or 0.8_SM1600, then if 2.0_ HN-72 is adopted, it is recommended that the DLA in the current *Bridge manual* be retained.

If 0.8_SM1600 is adopted, the opportunity should be taken to simplify the application of the DLA by making the value independent of span, and adopting the values in the AS5100. In the AS5100 the reduction in DLA at longer spans is dealt with by transitioning from the M1600 loading (DLA=0.3) to the S1600 loading (DLA=0).

D5.5 Horizontal loads

D5.5.1 Braking forces

Table A.38 in the literature review compares the magnitude of braking forces between various design codes, and indicates that although the factors are different between the *Bridge manual* and the AS5100, the final result is similar in terms of percentage of the vehicle weight. When calculating the braking load on long bridges, however, it is likely that the results will vary between the two bridge standards, as the *Bridge manual* requires 10% of the live load to be applied, and the AS5100 requires 15% of the live load to be applied.

With no international consensus on the correct formulation of the braking load, the researchers recommend that the existing Transport Agency approach be retained if a new vehicle loading standard utilises a loading model incorporating factored HN-72 loading, or if a loading model based on AS5100 SM1600 is adopted then the braking load calculations should confirm to the AS5100 approach. In the latter case, the vertical traffic load on which the braking load is based should be the factored values recommended in section D4.

D5.5.2 Centrifugal forces

As reported in the literature review, there is a common approach of all codes to base the centrifugal force as a proportion of the live load where the proportion is based on the following physics equation for centripetal acceleration, a:

$$a = \frac{v^2}{r}$$

where: v is the speed

r is the radius

D5.6 Load factors

D5.6.1 Serviceability limit state

The recommended loading model in section D4 has been formulated with a SLS load factor of 1.0. That is, the recommendations for the SLS loading are 2.0_HN-72 or 0.8_SM1600.

If the factored HN-72 loading is selected for the new vehicle loading standard, it is strongly recommended that the SLS loading is not described as a factor of HN-72 loading, in order to avoid confusion about when the factor is applied. The new SLS loading should be referred to by a new name (as HV loading, for example) with a factor of 1.0 at SLS.

D5.6.2 Ultimate limit state

The purpose of the ULS load factor is to reduce the probability of the load being exceeded to an acceptably low level. In practice in the Australian and New Zealand context it also includes, in part, an allowance for modelling uncertainty, geometric variability and such that is not fully allowed for in the capacity reduction factor that is used on the strength side of the design process. The calculation of an ultimate limit state load factor that provides an acceptable level of risk is not within the scope of this research project.

The current ULS load factors applied to the SLS loading are 1.67 for HN loading in the *Bridge manual* (1.49 for HO loading), and 1.8 in the AS5100.

The researchers have no basis for recommending alternative ULS load factors, and therefore recommend that the *Bridge manual* factor of 1.67 on the SLS live load (2.25 on the nominal live load) is retained if the new vehicle loading standard retains a loading model based upon HN-72 loading, or if a loading model based on AS5100 SM1600 is adopted a ULS load factor of 1.8 should be used.

D6   Evaluation loading

The evaluation loading is a design vehicle that is intended to replicate the structural actions from current legal vehicles, and can therefore be used in the assessment of bridges for current traffic loading. The current evaluation loading is 0.85_HN-72 (that is 85% of the 1972 HN loading model). This is an axle load of 102kN and a udl of 8.925kN/m.

The first part of this section reviews the applicability of 0.85_HN-72 for current legal loads. The following part proposes a new evaluation loading – both in terms of the *Bridge manual* HN-72 loading, and in terms of the AS5100 SM1600 loading.

No limit state load factor or DLA is included in this section dealing with the evaluation loading.

D6.1   Review of the 0.85_HN-72 evaluation loading

The evaluation loading of 0.85_HN-72 was introduced in *Assessment of posting weight limits for highway bridges* (Ministry of Works and Development 1974). Clause 6.1 of this document states:

> *For the determination of gross weight limits, the live load capacities need to be expressed as a percentage of the corresponding effect produced by the Class 1 loading of the Heavy Motor Vehicle Regulations. For the purpose of this instruction Class 1 loading shall be represented by 85% of the bridge design normal loading (HN-72).*

McGuire and Burt (1987) proposed that the heavy motor vehicle weight limits be increased (to the values that are in the current regulations for Class 1 vehicles). In their paper they reviewed the 0.85_HN-72 evaluation loading, and concluded that it should remain as the evaluation loading for the increased vehicle weight limits. The review was focused on simple spans up to 35m, with one legal vehicle on the span. The authors noted that the structural actions from single vehicle loading exceeded the design actions by up to 6.5% The authors gave general consideration to spans greater than 35m, concluding that 0.85_HN-72 loading would be exceeded by two legal vehicles at 6m headway for spans longer than 30m (moment exceeded) or 25m (shear exceeded). Continuous spans were not considered in the paper by McGuire and Burt.

The 0.85_HN-72 evaluation loading has served New Zealand well; however, it is exceeded by a significant margin at longer spans, and particularly so for continuous spans.

Figure D.48 plots single span bending moments up to 40m span. The bending moment from a single Class 1 vehicle with no coincident lane load is plotted (no udl) together with the bending moment from a Class 1 vehicle with coincident lane loading as defined in Annex D1 (with udl). The Class 1 vehicle is referred to as 'NZ2222 Class 1'. This is a truck and trailer combination that complies with the Class 1 mass and dimension requirements. The vehicle has a gross combination mass of 44t. From this graph, 0.85_HN-72 is a good representation of Class 1 legal loading over this span range.

However if the single span bending moments are plotted for spans longer than 40m, 0.85_HN-72 significantly underestimates the bending moments from Class 1 legal loading when coincident lane loading is allowed for (figure D.48). For continuous spans, 0.85_HN-72 underestimates the hogging moment at medium spans, but provides better agreement as the spans become longer (figure D.49). For shear, 0.85_HN-72 underestimates the shear from Class 1 legal loading for single and continuous spans longer than about 15m.

The complete set of graphs comparing 0.85_HN-72 to Class 1 legal loading are plotted in Annex D9.

Figure D.48   0.85_HN-72 sagging moment compared with Class 1 vehicle loading (40m and 100m span ranges)
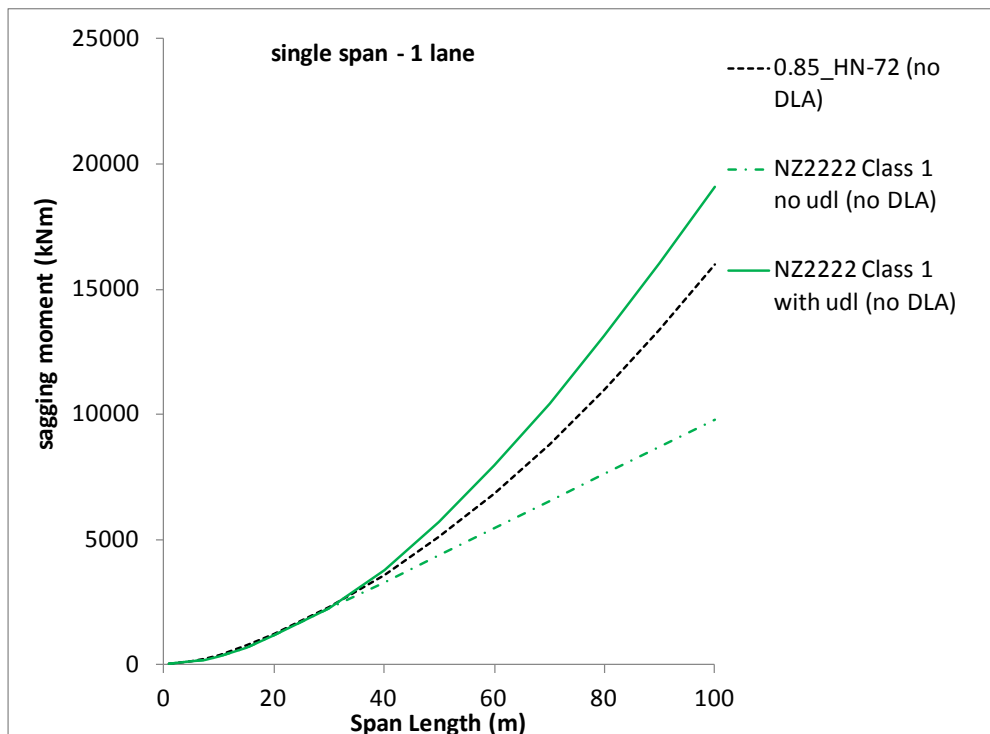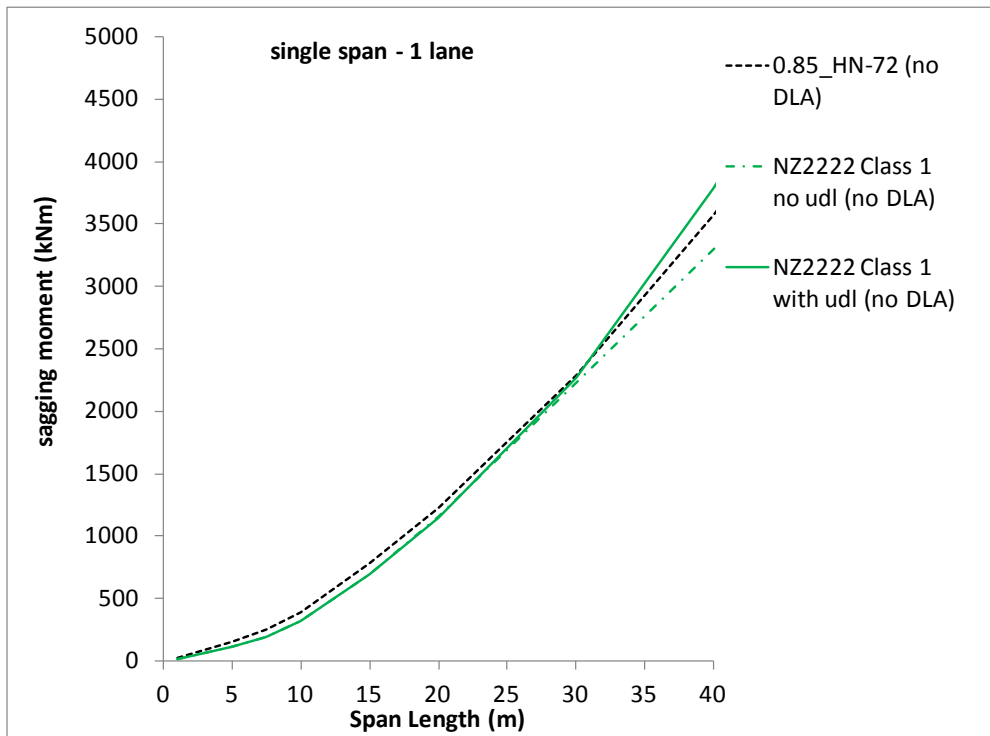
Figure D.49   0.85_HN-72 hogging moment compared with Class 1 vehicle loading (40m and 100m span ranges)
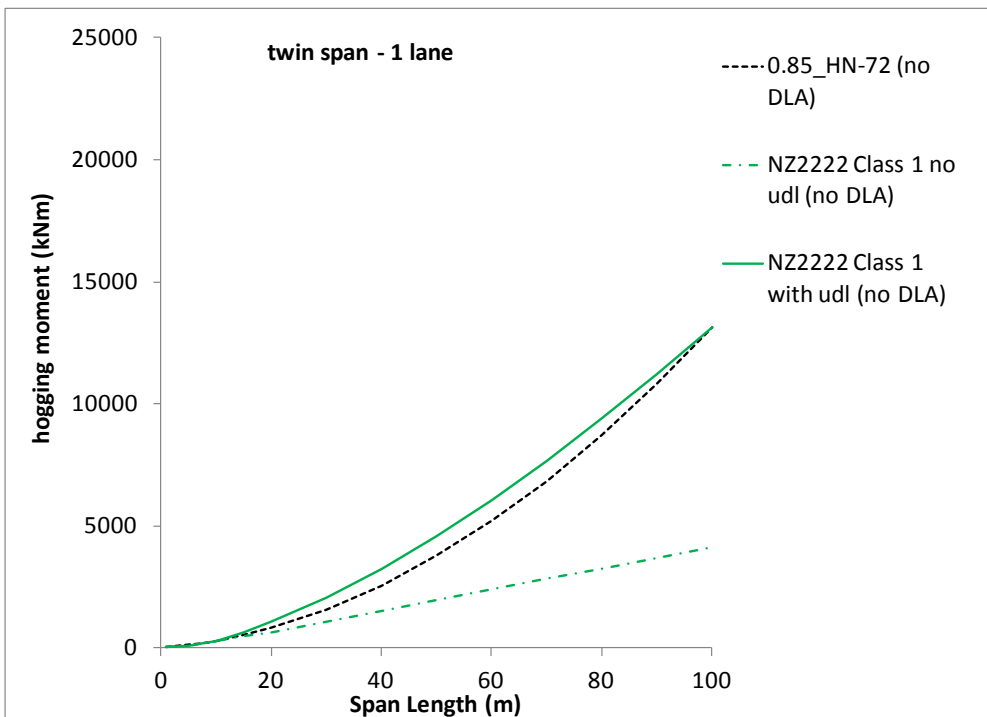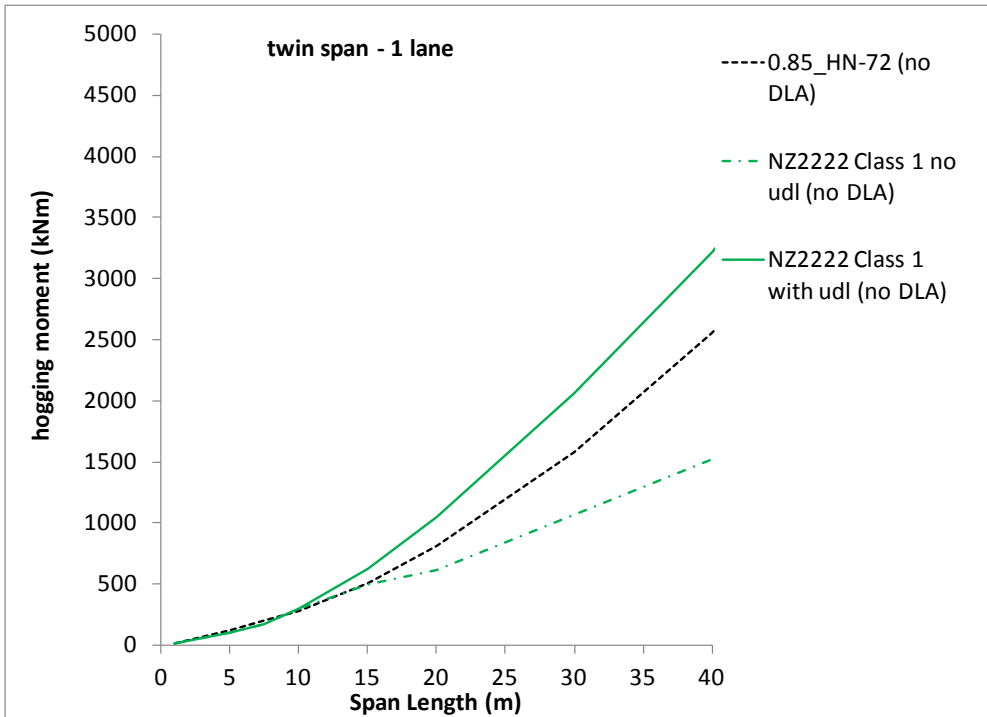
D6.2   Alternative evaluation loading

The previous section shows that the design actions from the 0.85_HN-72 loading are less than the structural actions from Class 1 legal vehicles over a wide range of spans.

In this section new evaluation loadings are proposed that provide a better match to the range of structural actions from Class 1 legal vehicles. Evaluation loadings are also proposed for HPMV vehicles. These evaluation loadings do not provide an upper bound to the actions from Class 1 and HPMV vehicles, rather they are a best fit to the actions over the span range from 0m to 100m. Therefore for some spans the evaluation loading underestimates some actions from Class 1 and HPMV vehicles. This approach was taken to ensure that the evaluation loading was not overly conservative for other spans and actions.

Evaluation loadings are proposed based on the *Bridge manual* HN-72 loading, and also on the AS5100 SM1600 loading.

D6.2.1 Limit state load factors

Serviceability or ULS load factors are not included in the discussion of an alternative evaluation loading.

D6.2.2 Application of a dynamic load allowance

As discussed in section D2.5.1, it is necessary to apply a DLA to the loads from the vehicles to enable comparison with the AS5100 design loading. The DLA applied in the *Bridge manual* has been used for this purpose.

D6.2.3 Evaluation loading based upon the *Bridge manual* HN-72 loading

Figure D.51 plots single span bending moments from Class 1 and HPMV vehicles with coincident lane loading. The proposed evaluation loading is 1.0_HN-72 for Class 1 vehicle loading, and 1.1_HN-72 for HPMV loading. Figure D.52 plots the same information for hogging moments in twin continuous spans.

The complete set of graphs for the proposed evaluation loading based on the *Bridge manual* HN-72 loading are plotted in annex D10.

The ratio of the proposed evaluation loading to the vehicle loading is tabulated in table D.16 and plotted in figure D.50.

Table D.16 Ratio of design actions: (1.0_HN-72)/(Class 1) and (1.1_HN-72)/(HPMV)

Span	(1.0_HN-72)/(Class 1)			(1.1_HN-72)/(HPMV)		
Metres	Sagging moment	Shear	Hogging moment	Sagging moment	Shear	Hogging moment
5	1.64	1.39	1.41	1.35	1.18	1.43
7.5	1.56	1.47	1.39	1.34	1.35	1.21
10	1.47	1.42	1.10	1.39	1.37	1.11
15	1.32	1.29	**0.96**	1.41	1.26	**0.99**
20	1.25	1.19	**0.91**	1.27	1.17	1.00
30	1.18	1.08	**0.90**	1.14	1.06	**0.91**
40	1.11	**0.98**	**0.94**	1.08	**0.99**	**0.94**
50	1.05	**0.93**	**0.97**	1.04	**0.93**	**0.98**
60	1.01	**0.91**	1.01	1.01	**0.92**	1.02
70	**0.99**	**0.90**	1.05	**0.99**	**0.91**	1.07
80	**0.98**	**0.90**	1.09	**0.98**	**0.91**	1.11
90	**0.98**	**0.91**	1.13	**0.98**	**0.91**	1.16
100	**0.99**	**0.91**	1.17	**0.99**	**0.93**	1.20

Figure D.50   Ratio of design actions: (1.0_HN-72)/(Class 1) and (1.1_HN-72)/(HPMV)
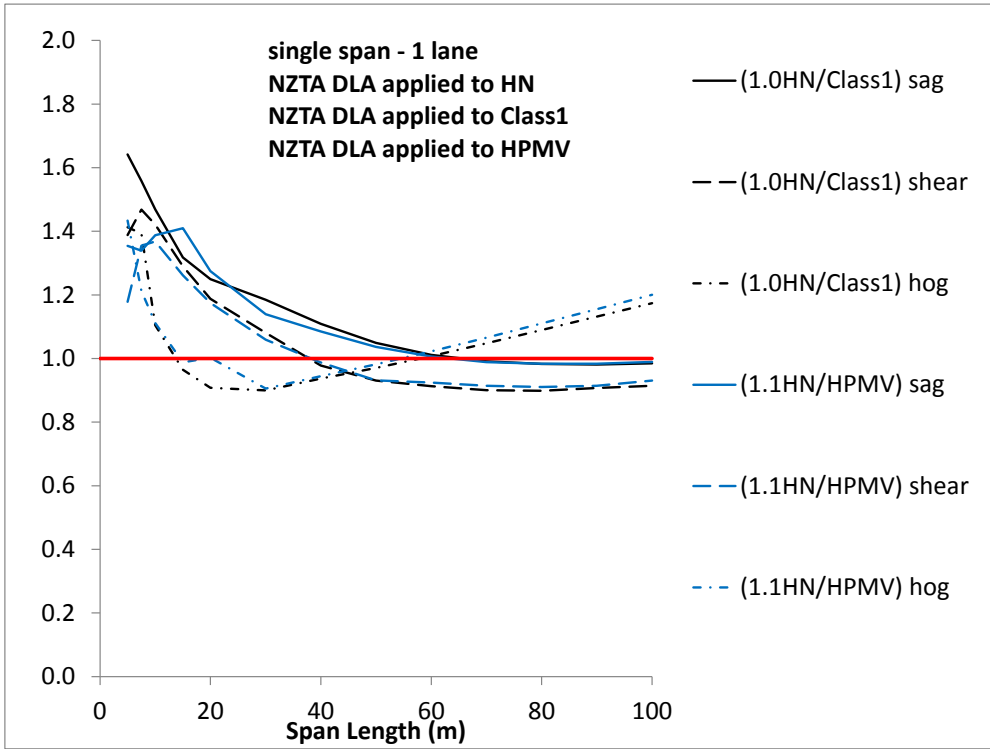
Figure D.51   Proposed HN evaluation loading plotted against legal vehicle loading – sagging moment

Figure D.52 shows the single span - 1 lane chart with sagging moment (kNm) on the y-axis and Span Length (m) on the x-axis.
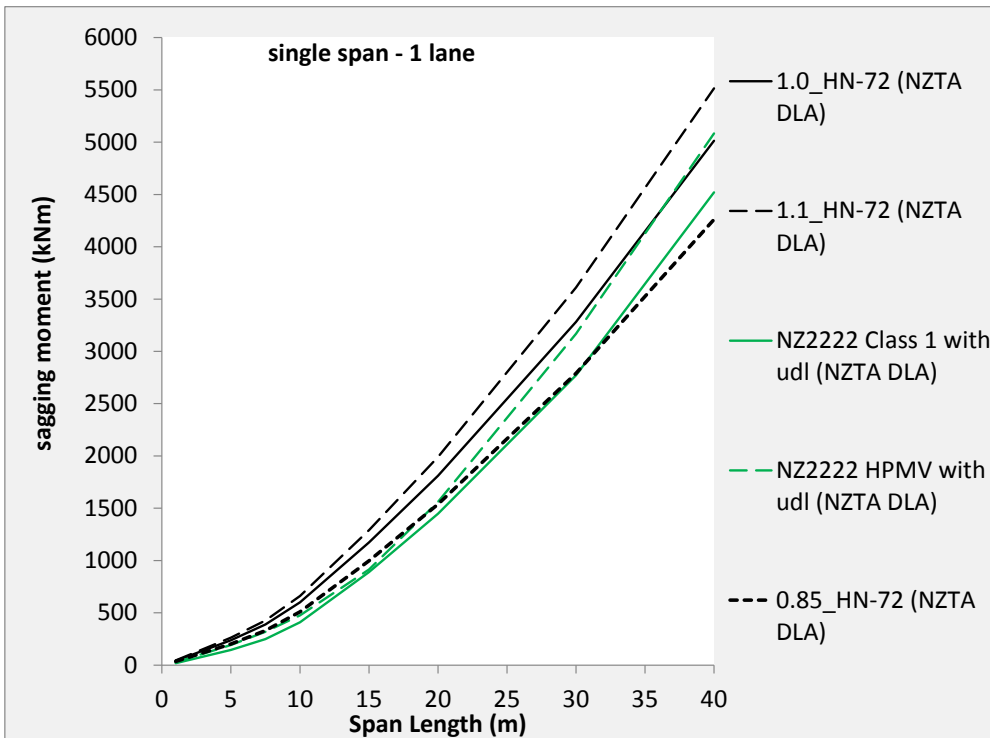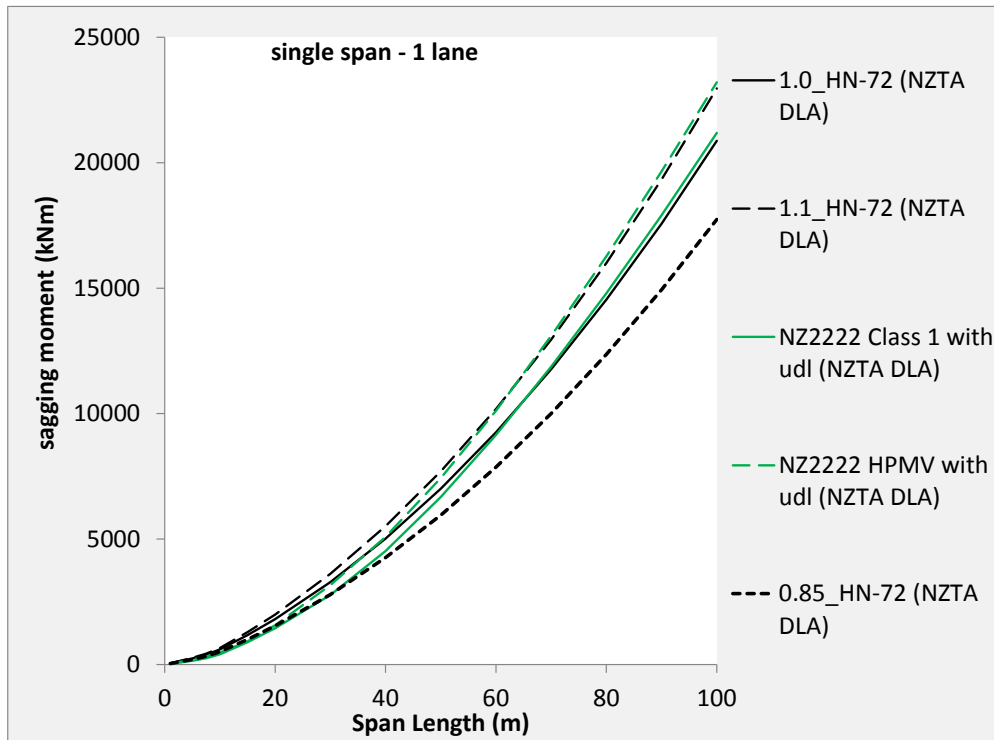
Figure D.52 **Proposed HN evaluation loading plotted against legal vehicle loading – hogging moment**
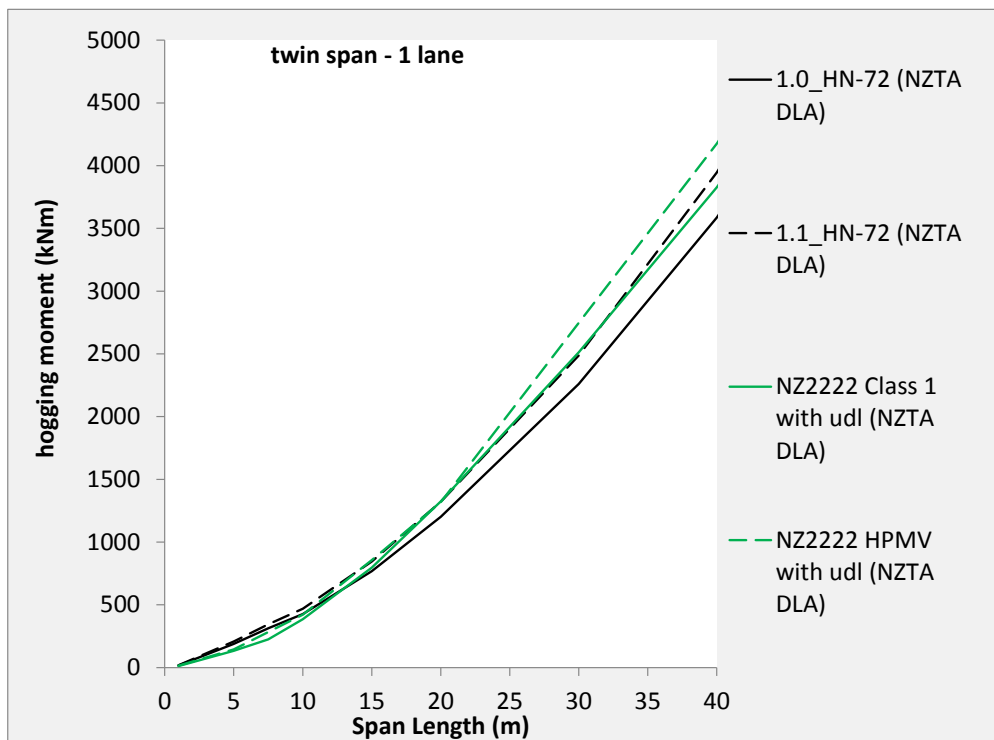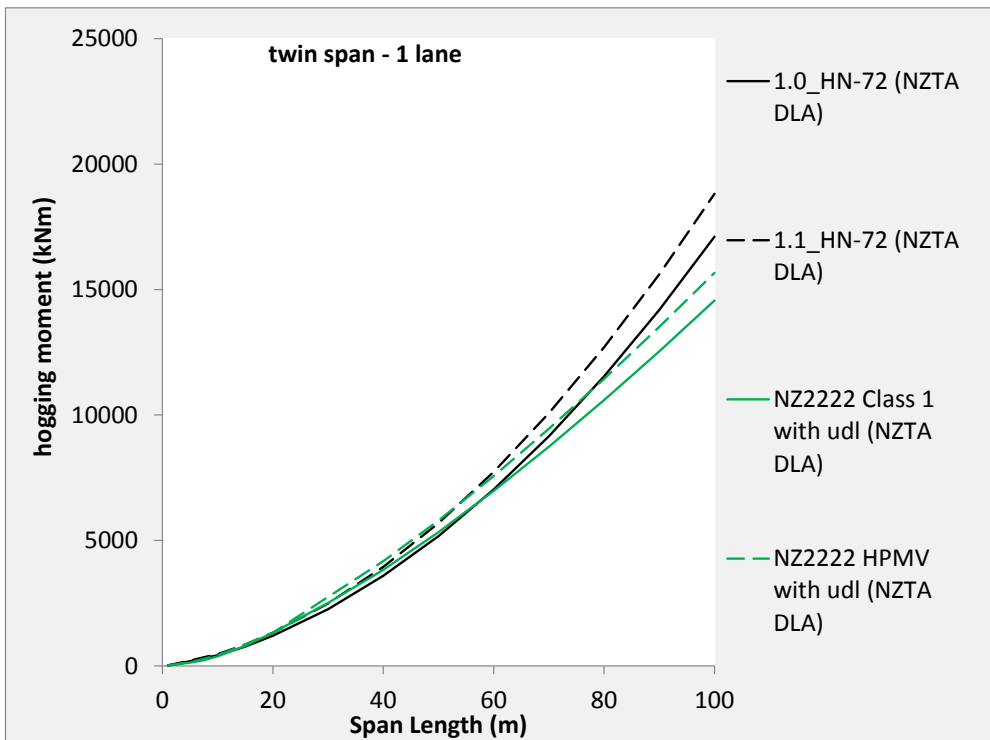
D6.2.4 Evaluation loading based upon the AS5100 SM1600 loading

Figure D.54 plots single span bending moments from Class 1 and HPMV vehicles with coincident lane loading. The proposed evaluation loading is 0.40_SM1600 loading for Class 1 vehicle loading and 0.45_SM1600 for HPMV loading. Figure D.55 plots the same information for hogging moments in twin equal spans.

The proposed evaluation loading models of 0.40_SM1600 and 0.45_SM1600 do not include the W80 (8.1t) wheel load or the A160 (16.3t) axle load because these are considered to be excessive for the purpose of evaluation. 0.40_SM1600 includes an axle load of 48kN (4.9t) within the M1600 vehicle. However for very short elements including small culverts and bridge decks, an axle load of 48kN may be non-conservative for evaluation. Therefore the proposed evaluation loading models of 0.40_SM1600 and 0.45_SM1600 include a 100kN axle loading as an additional requirement.

The complete set of graphs for the proposed evaluation loading based on the AS5100 SM1600 loading are plotted in annex D10.

The ratio of the proposed evaluation loading to the vehicle loading is tabulated in table D.17 and plotted in figure D.53.

The table and graphs indicate that the proposed evaluation loadings of 0.40_SM1600 loading for Class 1 vehicle loading and 0.45_SM1600 for HPMV loading underestimate the shear by up to 24% at long spans. This is largely due to the application of the DLA – for shear the *Bridge manual* applies a DLA of 0.3 which does not decrease at long spans.

Table D.17   Ratio of design actions: (0.40_SM1600)/(Class 1) and (0.45_SM1600)/(HPMV)

Span	(0.40_SM1600)/(Class 1)			(0.45_SM1600)/(HPMV)		
Metres	Sagging moment	Shear	Hogging moment	Sagging moment	Shear	Hogging moment
5	1.21	1.12	1.04	1.00	**0.98**	1.08
7.5	1.23	1.00	1.20	1.08	**0.94**	1.07
10	1.09	1.07	1.03	1.05	1.05	1.06
15	1.04	1.02	1.12	1.14	1.02	1.17
20	1.03	1.04	1.05	1.07	1.06	1.19
30	1.12	1.08	**0.94**	1.10	1.08	**0.97**
40	1.16	1.00	**0.89**	1.16	1.03	**0.92**
50	1.12	**0.94**	**0.90**	1.13	**0.96**	**0.93**
60	1.06	**0.89**	**0.93**	1.08	**0.92**	**0.97**
70	1.02	**0.84**	**0.97**	1.04	**0.87**	1.00
80	**0.98**	**0.80**	1.00	1.01	**0.83**	1.04
90	**0.95**	**0.78**	1.04	**0.98**	**0.80**	1.08
100	**0.95**	**0.76**	1.08	**0.97**	**0.79**	1.13

Figure D.53   Ratio of design actions: (0.40_SM1600)/(Class 1) and (0.45_SM1600)/(HPMV)
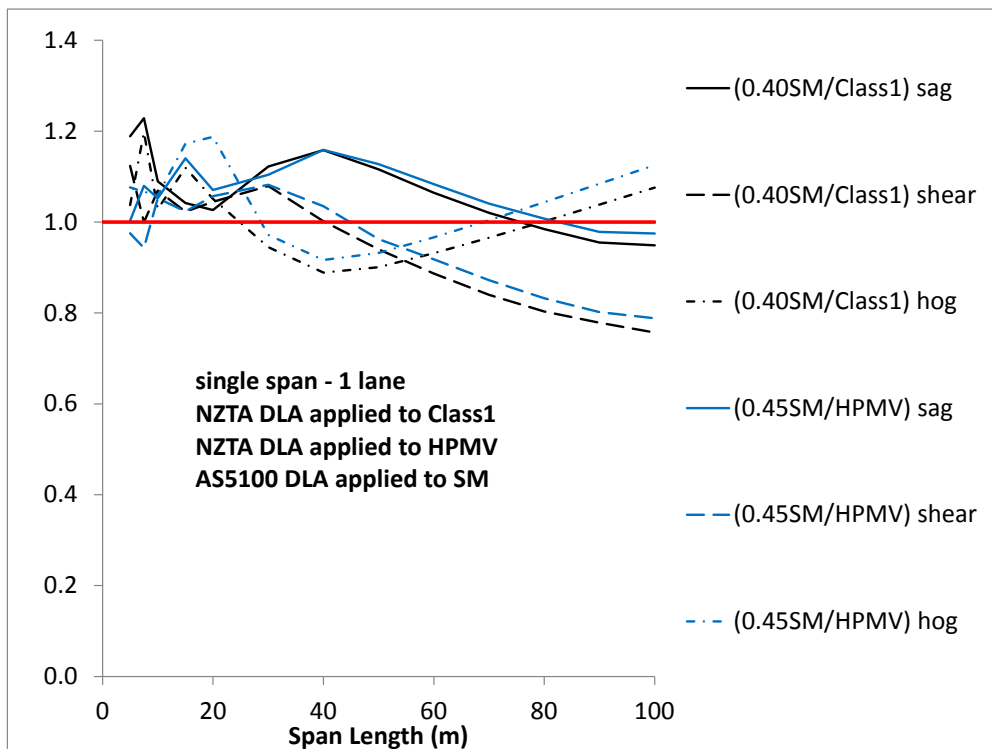
Figure D.54   Proposed SM1600 evaluation loading plotted against legal vehicle loading – sagging moment
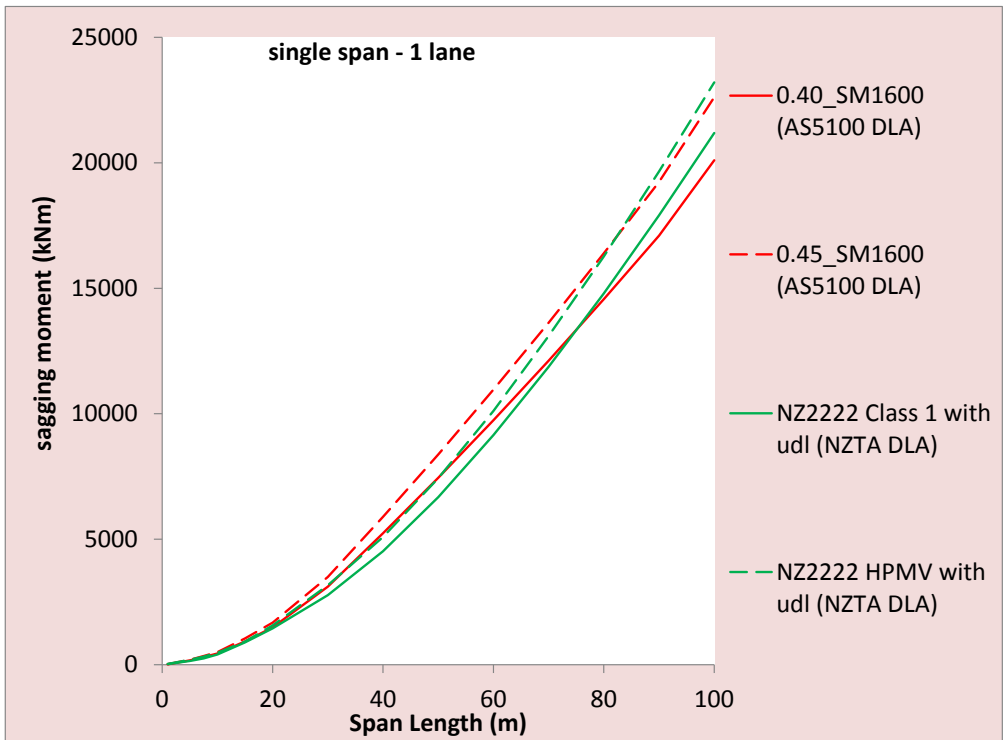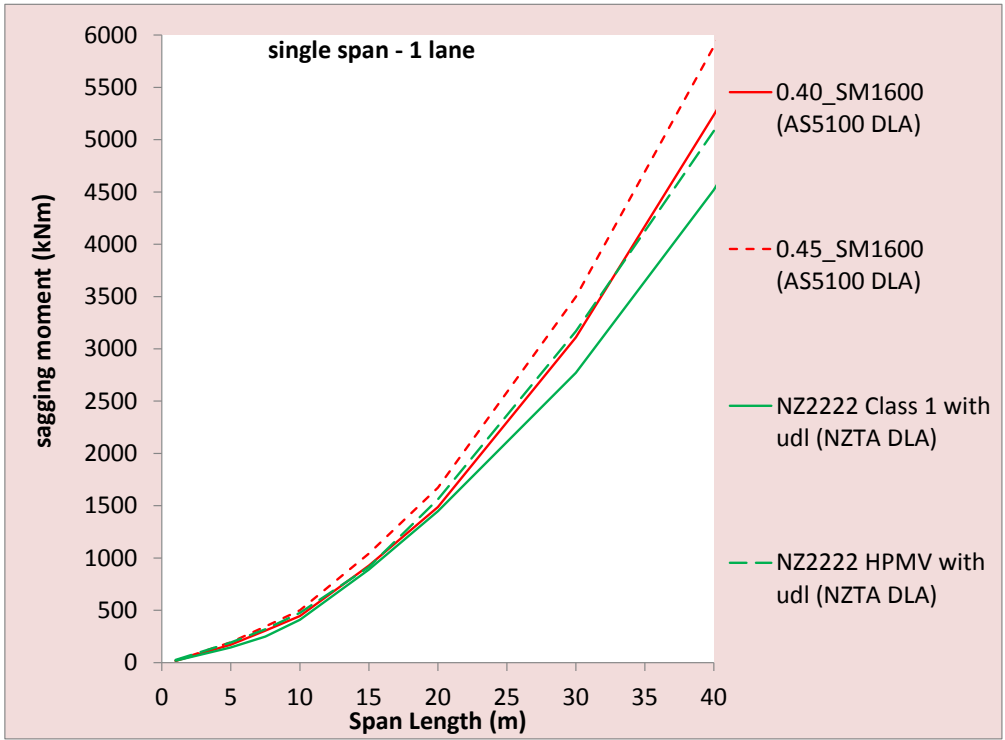
Figure D.55   Proposed SM1600 evaluation loading plotted against legal vehicle loading – hogging moment
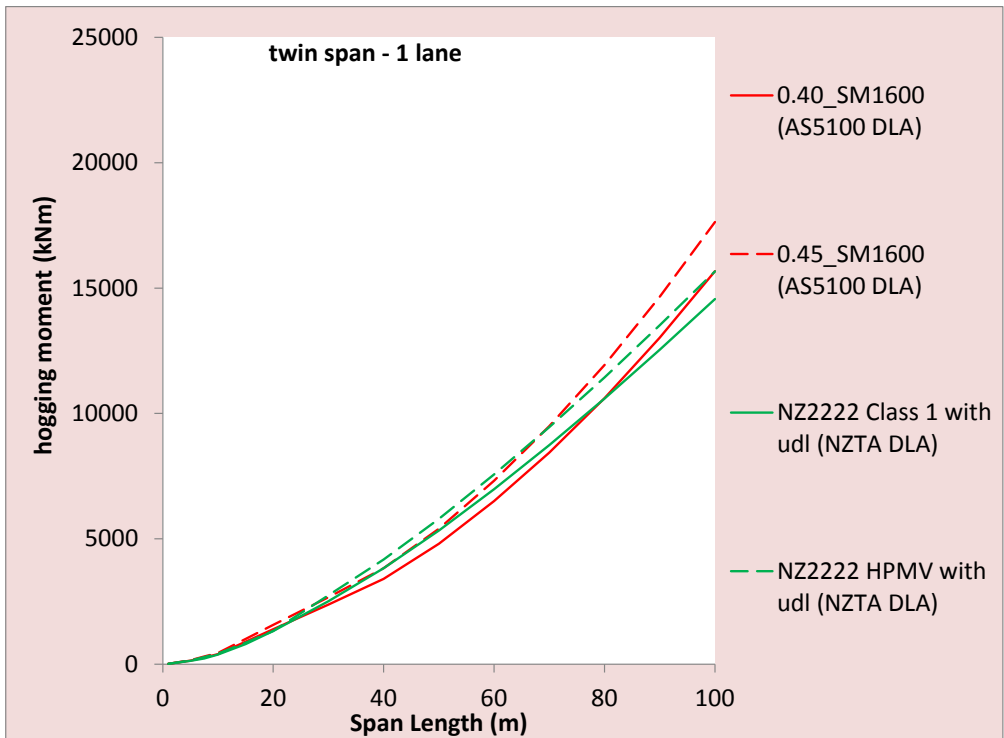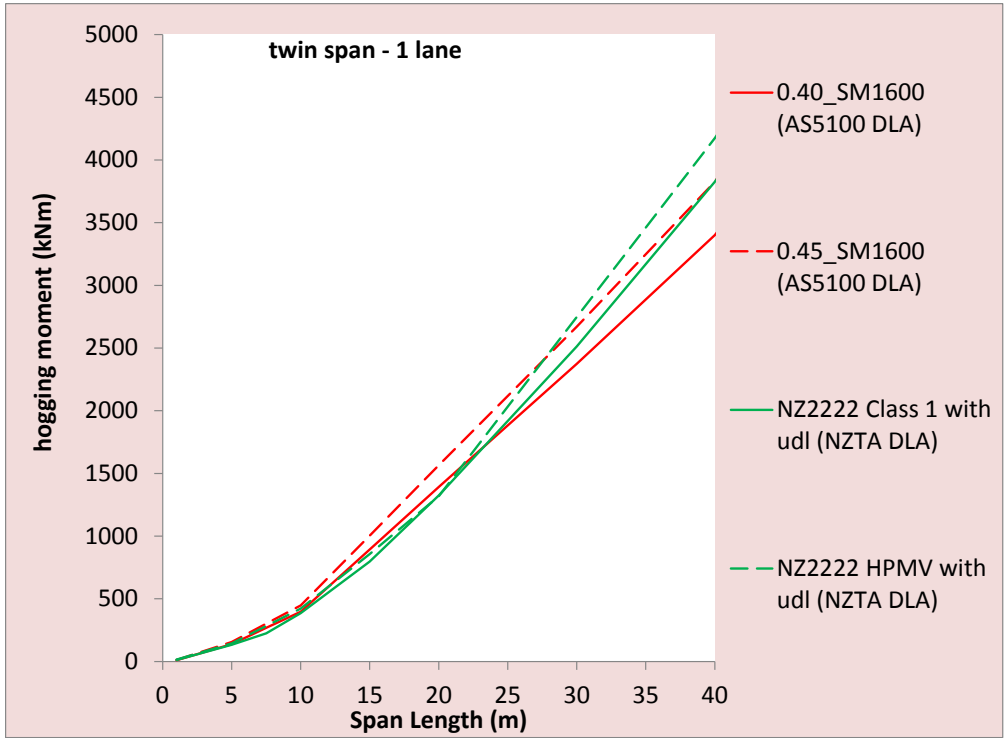
D6.2.5   Recommended evaluation loading

Notwithstanding that the 0.85_HN-72 loading has served New Zealand well, if the evaluation loading is to encompass a complete range of spans and design actions the researchers recommend that a new evaluation loading be adopted as presented in table D.18.

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The recommended evaluation vehicle loading model for Class 1 legal vehicles is 0.40_SM1600 and consists of:

- 40% of the M1600 moving load

- 40% of the S1600 stationary load

- 100kN axle load

- a DLA of 0.4 for the axle load, 0.35 for the tri-axle set in M1600, 0.3 for the complete M1600, and 0.0 for S1600

The recommended evaluation loading model for HPMV legal vehicles is 0.45_SM1600 and consists of:

- 45% of the M1600 moving load

- 45% of the S1600 stationary load

- 100kN axle load

- a DLA of 0.4 for the axle load, 0.35 for the tri-axle set in M1600, 0.3 for the complete M1600, and 0.0 for S1600

Table D.18   Recommended evaluation loading

Legal loading	Recommended evaluation loading	
	Based upon the *Bridge manual* HN-72 loading	Based upon the AS5100 SM1600 loading
Class 1 vehicles	1.0_ HN-72	0.40_SM1600 (includes 100kN axle)
HPMV vehicles	1.1 _HN-72	0.45_SM1600 (includes 100kN axle)

D6.3   Impact on existing bridge evaluations

The proposed evaluation load of 0.40_SM1600 for Class 1 vehicles is an increase over the current evaluation load of 0.85_HN-72, for most spans and design actions. This is because, as explained in section D6.1, the current evaluation load underestimates the structural actions. The ratio of 0.40_SM1600 to 0.85_HN-72 is plotted in figure D.56. This will lead to existing bridges having a reduced load rating if they are re-rated against 0.40_SM1600.

However, for sagging moment in single-span bridges in the span range from 10m to 30m, the difference between 0.40_SM1600 and 0.85_HN-72 is modest. This is plotted in figure D.57, and shows that the design actions from 0.40_SM1600 are between 11% greater and 7% less than the design actions from 0.85_HN-72. As the majority of bridges fall into this category, the impact of changing the evaluation load will be relatively minor.

Figure D.56   Ratio of design actions: (0.40_SM1600/0.85_HN)
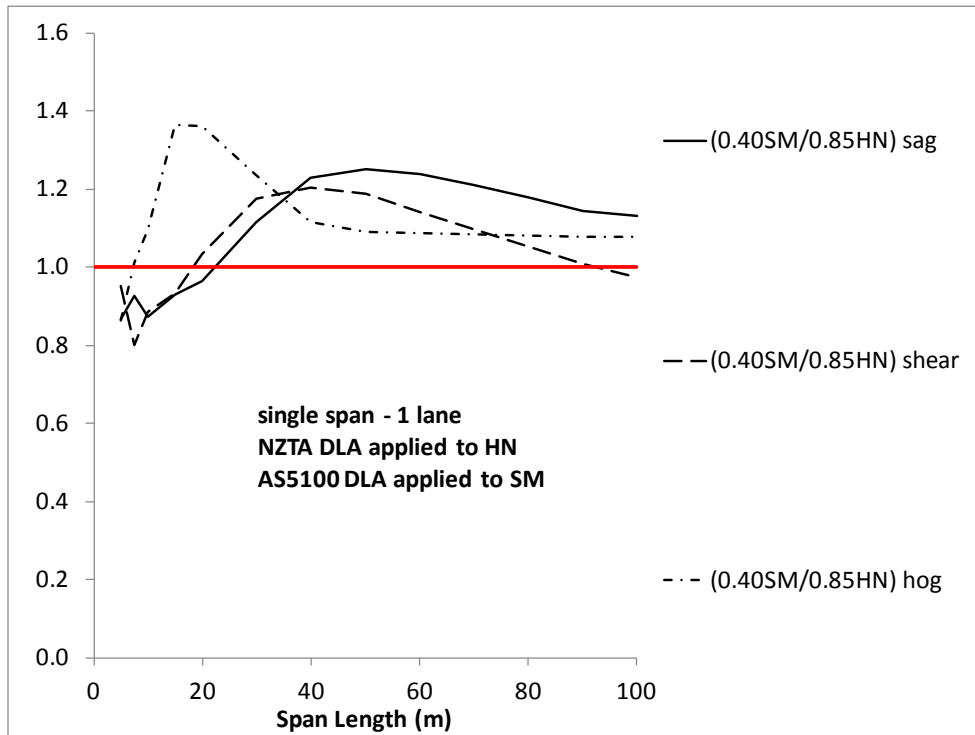
Figure D.57   Ratio of sagging moment up to 30m simple span: (0.40_SM1600/0.85_HN)
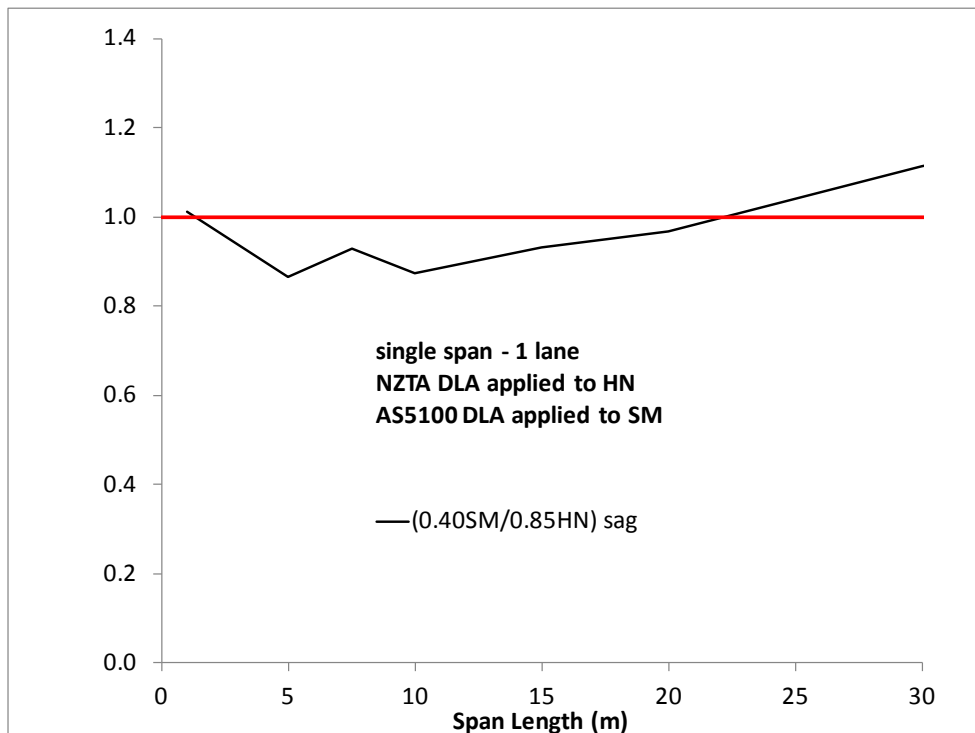
D7   References

de Pont, J (2008) Optimising log truck configurations. *10th International Symposium on Heavy Vehicle Transport Technology*, Paris.

Heywood, R, R Gordon and G Boully (2000) Australia's bridge design load model: planning for an efficient road transport industry. *Transport research record 1696.*

McGuire, J and E Burt (1987) Changes to heavy motor vehicle weight limits. *New Zealand Roading Symposium 4.*

Ministry of Works and Development (1974) *Assessment of posting weight limits for highway bridges.*

Roberts, W and R Heywood (2001) Bridge evaluation. Transit NZ Heavy Vehicle Limits Project.

Sleath, L (1995) Investigation into the feasibility of heavy transport routes in New Zealand. *Road Transport Technology 4*, Ann Arbor USA.

Sweatman, P, R Di Cristoforo and D Kalasih (2004) Benchmarking heavy vehicle productivity and safety performance. *8th International Symposium on Heavy Vehicle Weights and Dimensions*, Gauteng, South Africa 2008.

Wanty, D and L Sleath (1998) Further Investigations into the feasibility of heavy transport routes in New Zealand. *5th International Symposium on Heavy Vehicle Weights and Dimensions*, Brisbane, 1998.

Annex D1: Calculation of the coincident lane loading

When determining the structural actions due to a specific vehicle (for example a vehicle complying with current Class 1 loading) it is necessary to place additional loading in the same traffic lane to represent other vehicles that are travelling in front of and behind the specific vehicle. In this report this loading is referred to as the coincident lane loading.

Two methods for calculating the coincident lane loading were presented in section D3.3.

- Based upon the WiM data analysis in section D3.3, a variable udl for sagging bending and shear of:

$$udl = 140 \times span^{-0.5} \; kN/m$$

    and for hogging bending of:

$$udl = 109 \times span^{-0.54} \; kN/m$$

- Based upon the load intensity formula in section D3.3, a concentrated load and a fixed udl:

$$concentrated\ load\ of\ 400\ kN\ and\ a\ udl = 10\ kN/m$$

The bending moment and shear forces due to the R22T22 HPMV truck were calculated with a coincident lane loading based upon both approaches. The bending moments and shear forces for single and twin continuous spans are plotted below. They show that both approaches to the calculation of a coincident lane loading provided similar results.

In this report, the coincident lane loading is based upon data from the WiM analysis.

Graphs on the left are for spans 0 to 100m, and on the right for 0 to 40m.
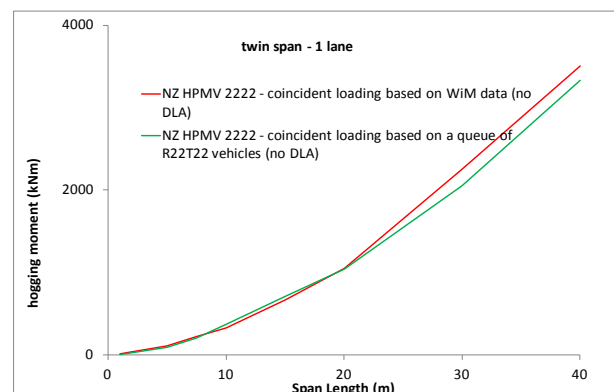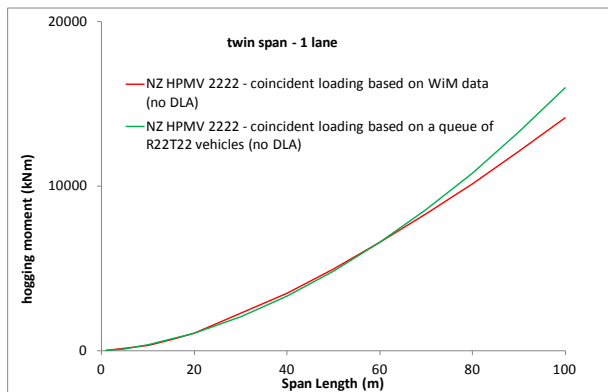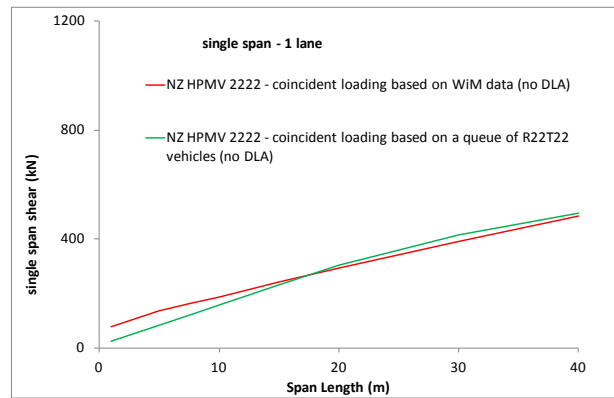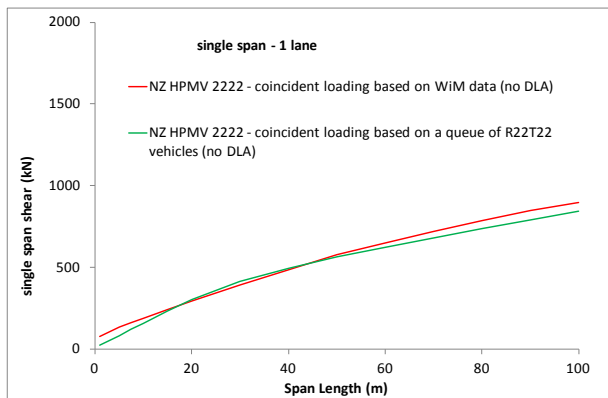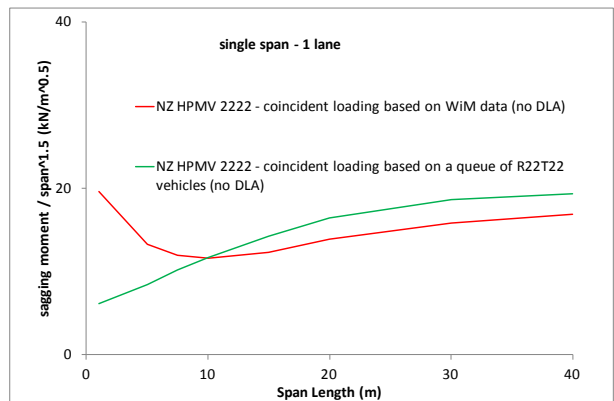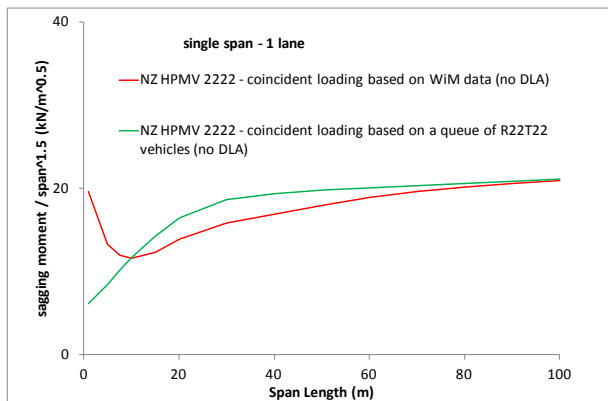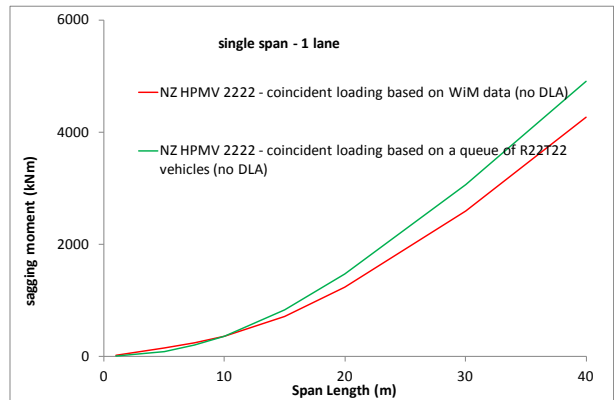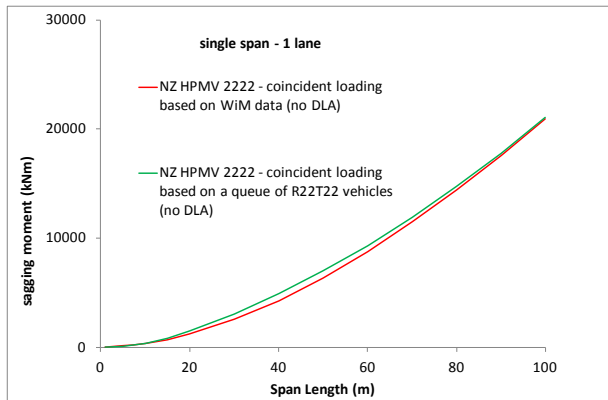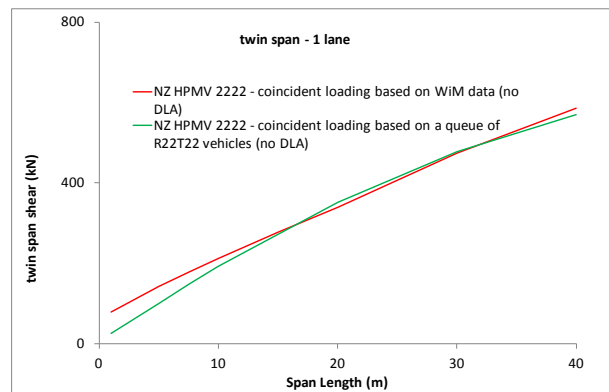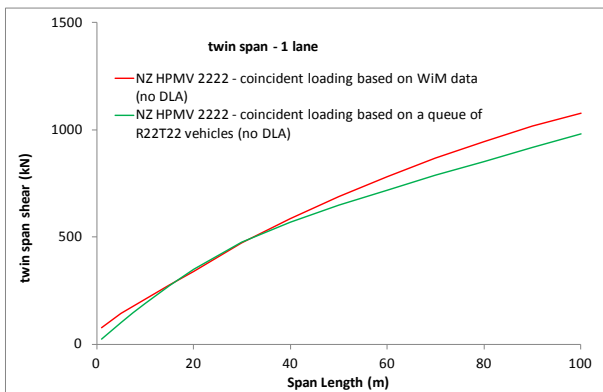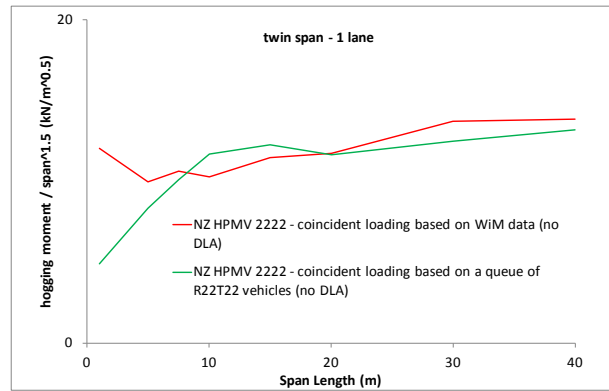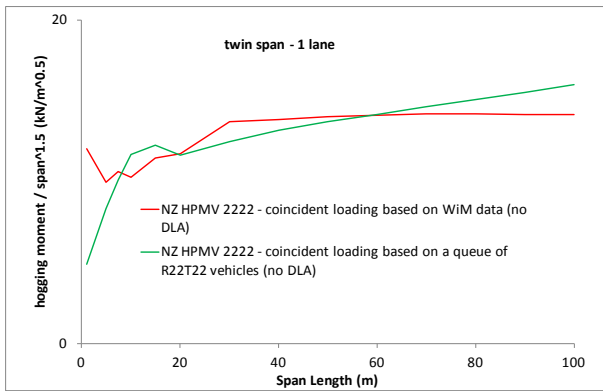
Annex D2: Structural actions from the WiM analysis

Structural actions for single and twin continuous spans up to 100m are plotted for the three WiM loading scenarios – recorded weight and spacing of traffic, 1% queued and 4% queued. In addition the 95th percentile values of the daily maximum structural actions are plotted (as used for calculating the coincident lane loading).

Graphs on the left are for spans 0 to 100m, and on the right for 0 to 40m.
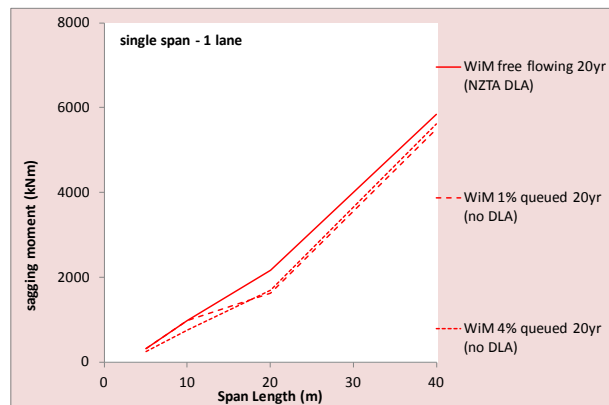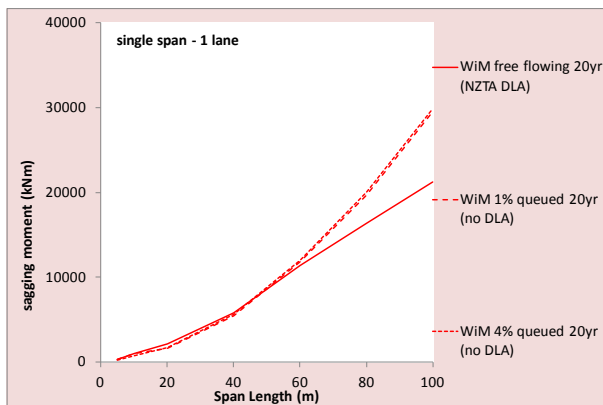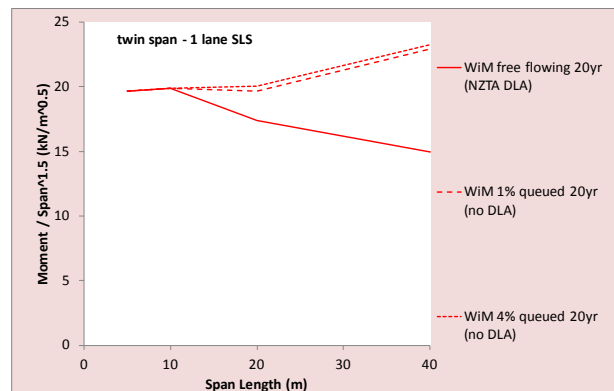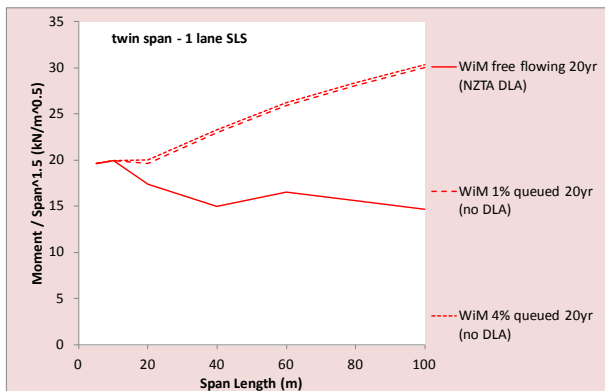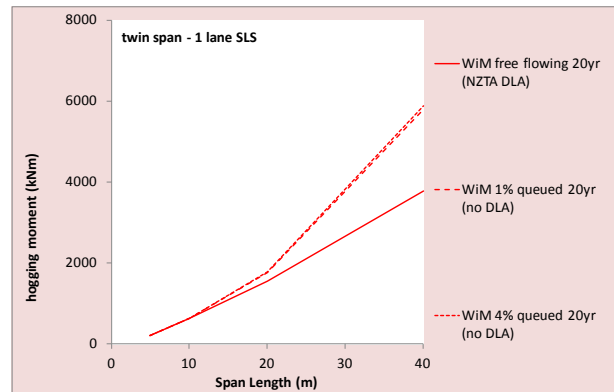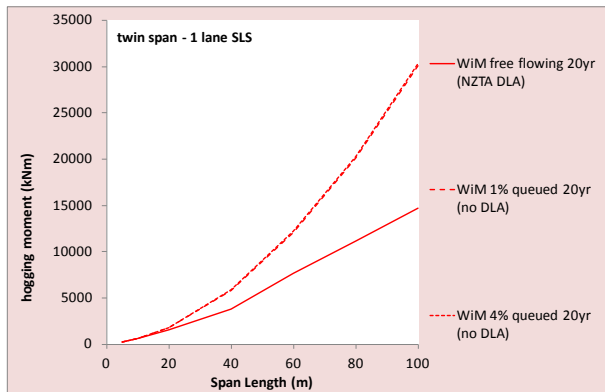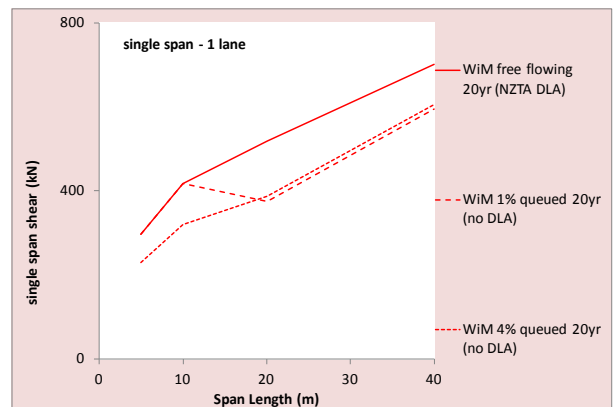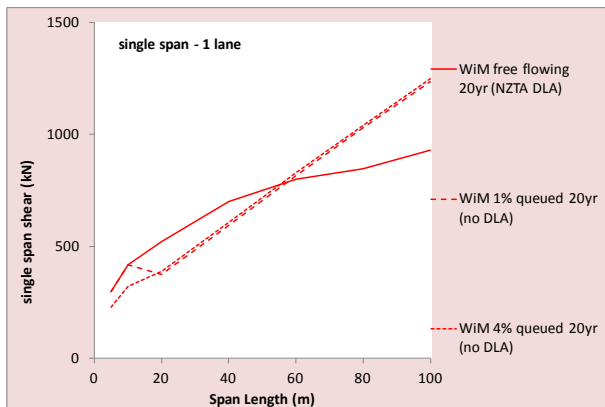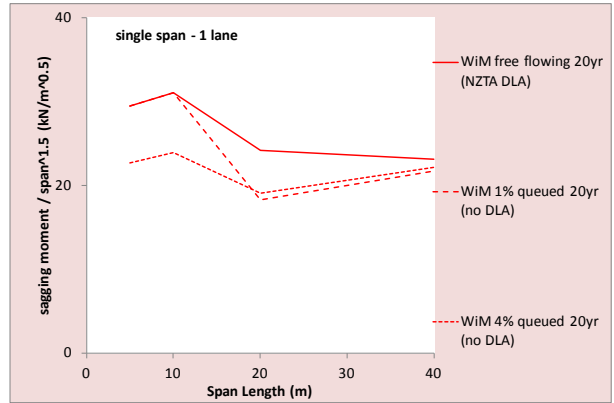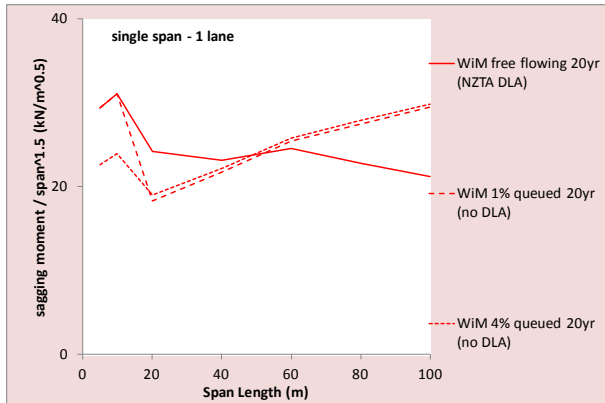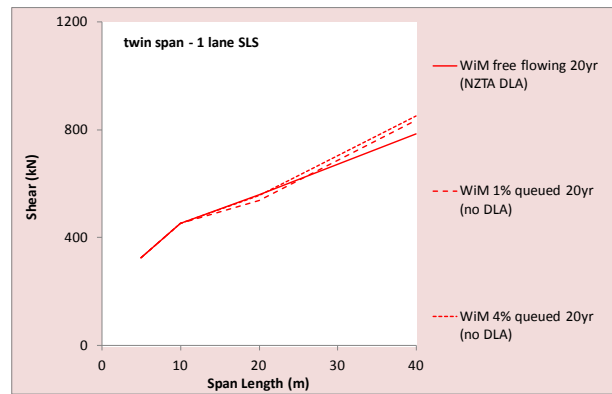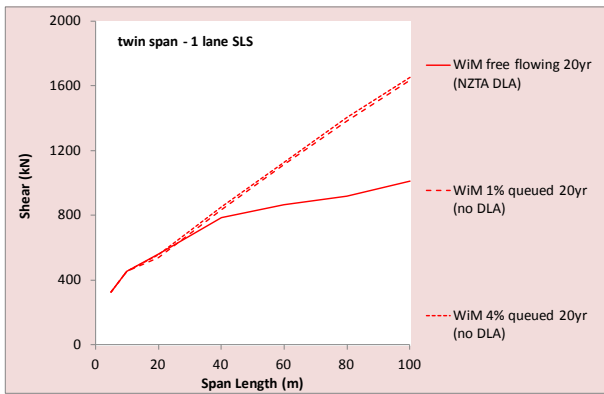
Annex D3: Structural actions from legal vehicles

Structural actions for single and twin continuous spans up to 100m were plotted for 8 representative legal vehicles from New Zealand, Australia and Europe. Coincident lane loading was applied in each case. Detailed axle mass and spacing information is provided for each vehicle in table D.7.

Graphs on the left are for spans 0m to 100m, and on the right for 0m to 40m.
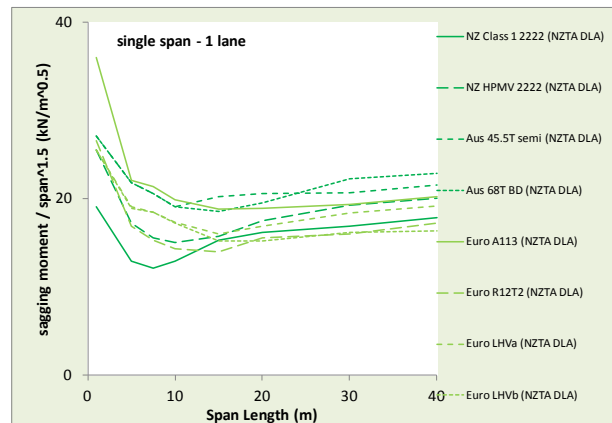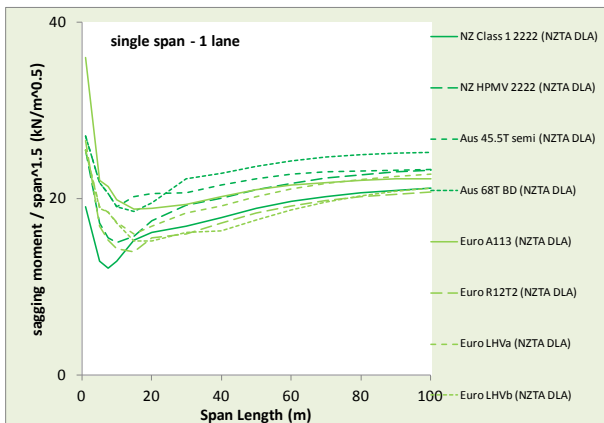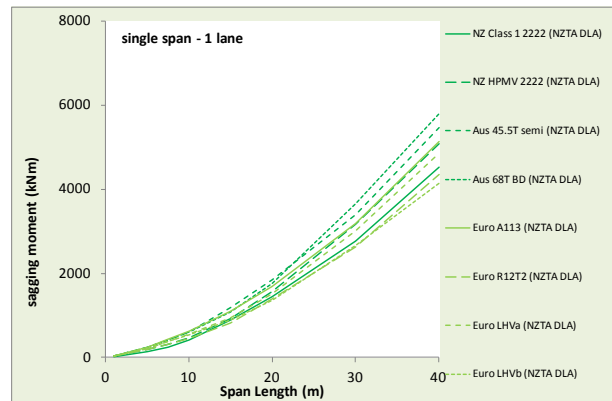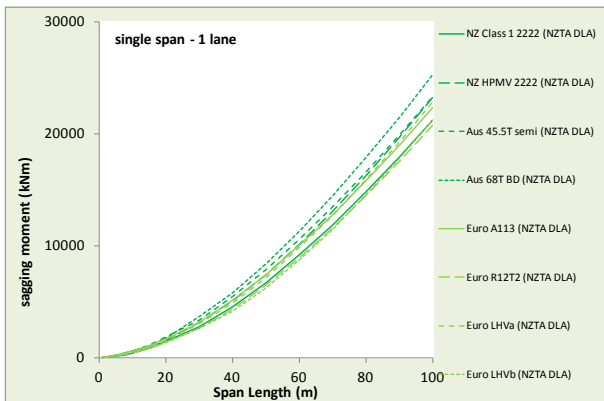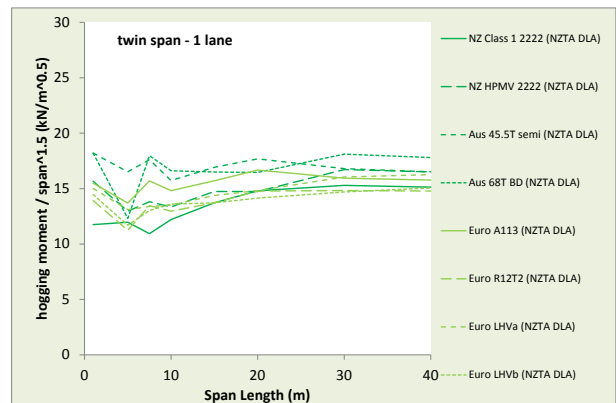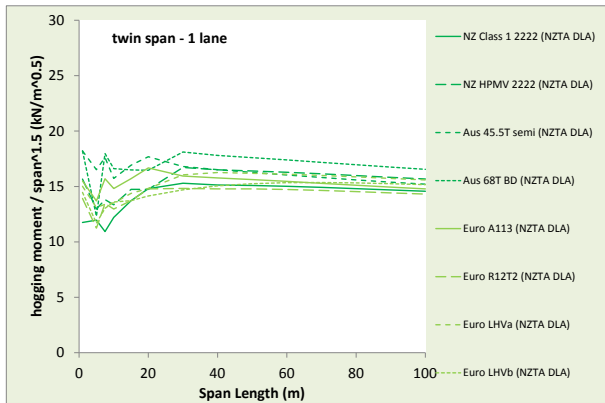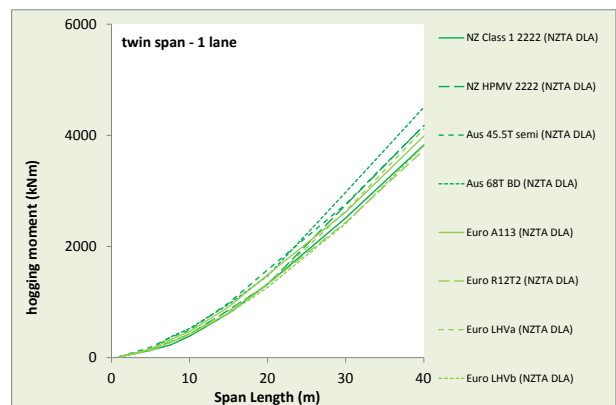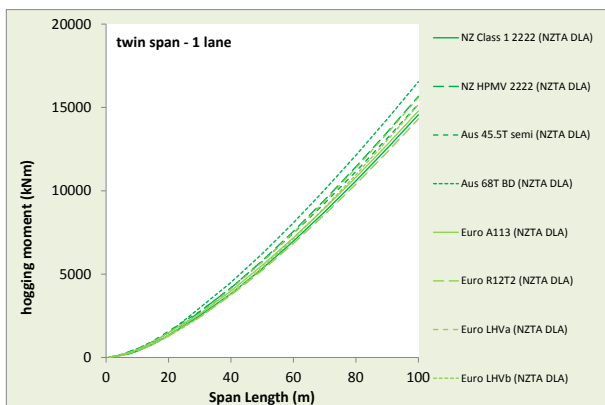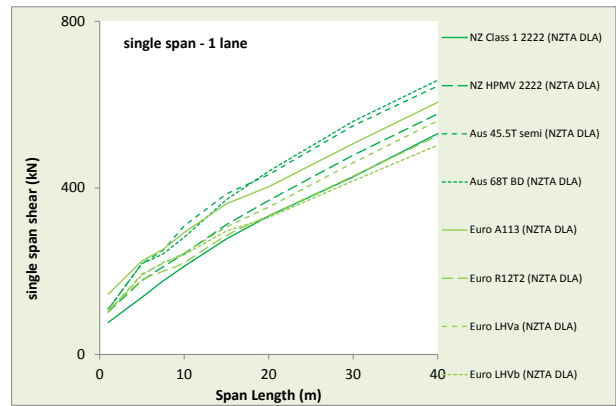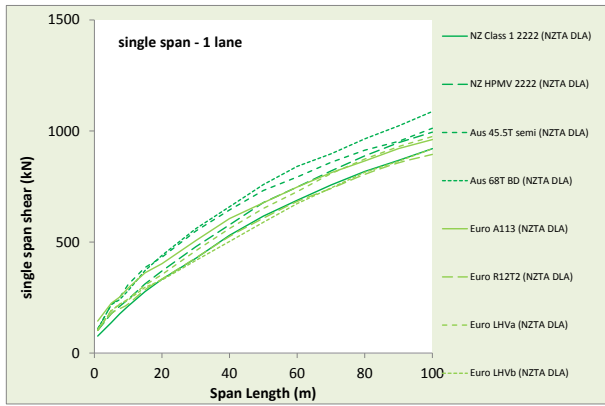
Annex D4: Structural actions from AWLAG questionnaire vehicles

Structural actions for single and twin continuous spans up to 100m were plotted for eight vehicles included in responses to the AWLAG questionnaire. Coincident lane loading was applied in each case. Detailed axle mass and spacing information is provided for each vehicle in table D.8.

Graphs on the left are for single spans, and on the right for twin continuous spans.
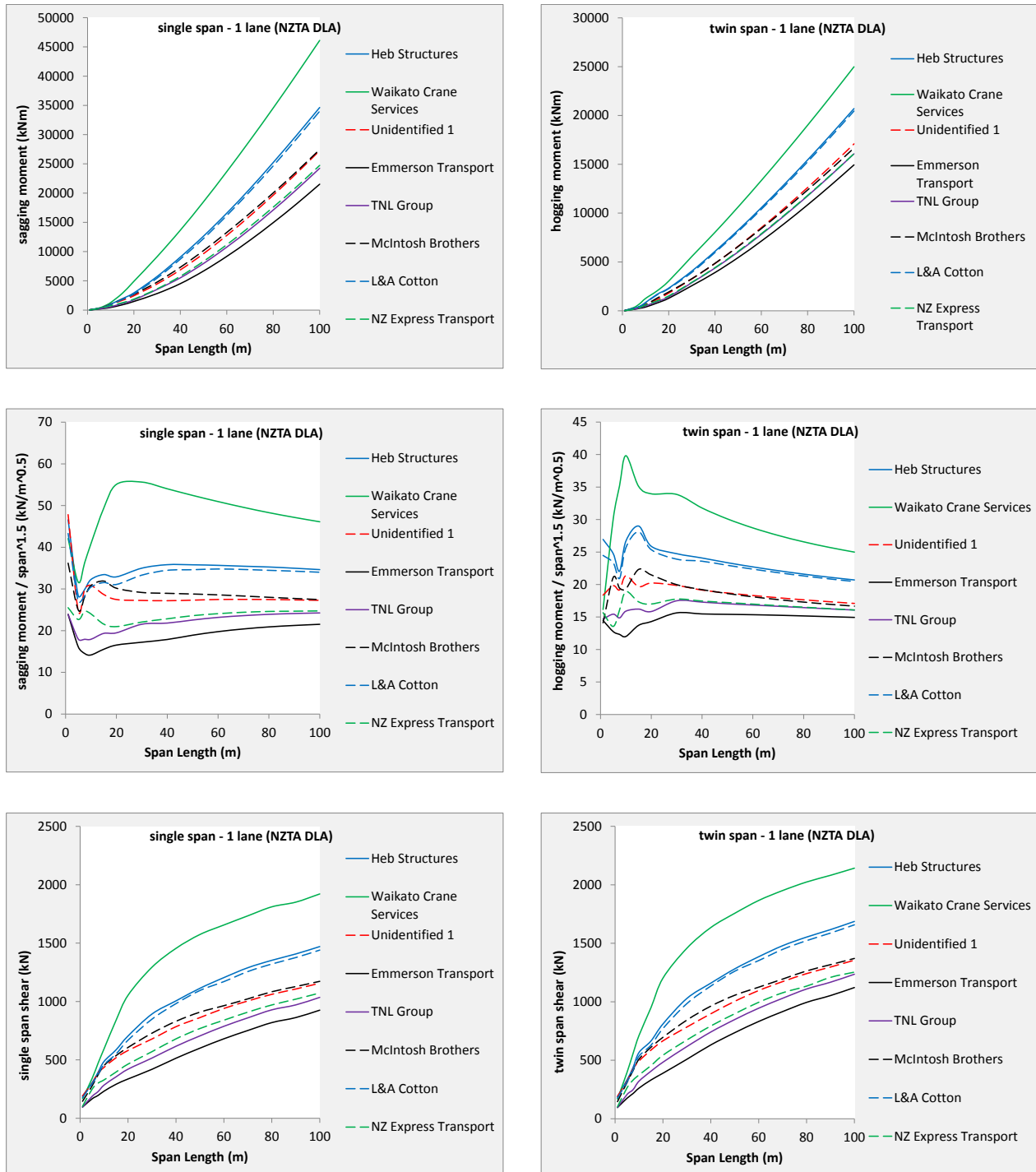
Annex D5: Structural actions from AWLAG responses, permit vehicles and cranes

Structural actions for single and twin continuous spans up to 100m were plotted for the envelope of maximum structural actions from the AWLAG questionnaire response vehicles, the envelope of maximum structural actions from the Transport Agency permit applications, and the structural actions from the three European cranes. Coincident lane loading is included in all cases.

Graphs on the left are for spans 0 to 100m, and on the right for 0 to 40m.

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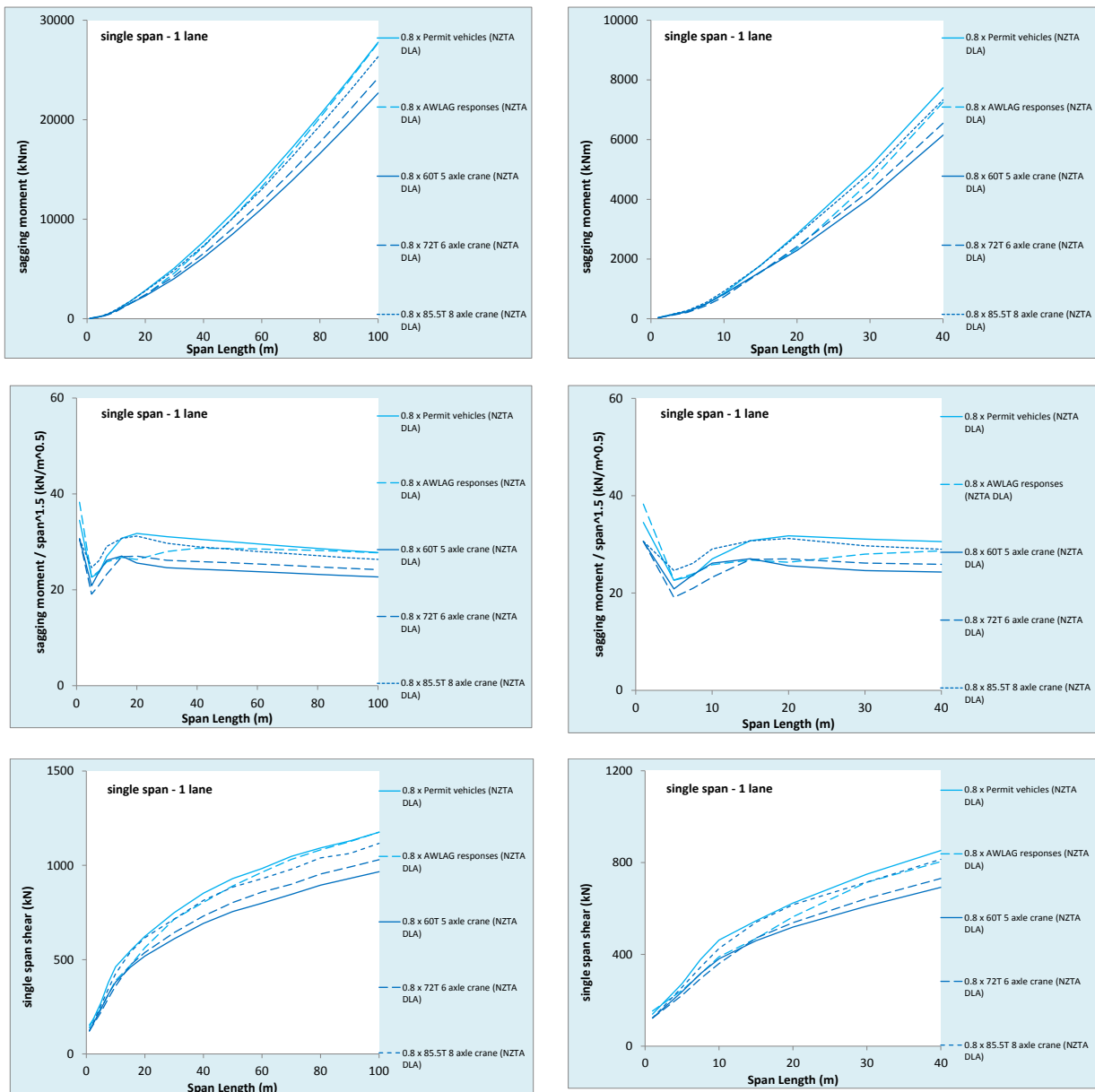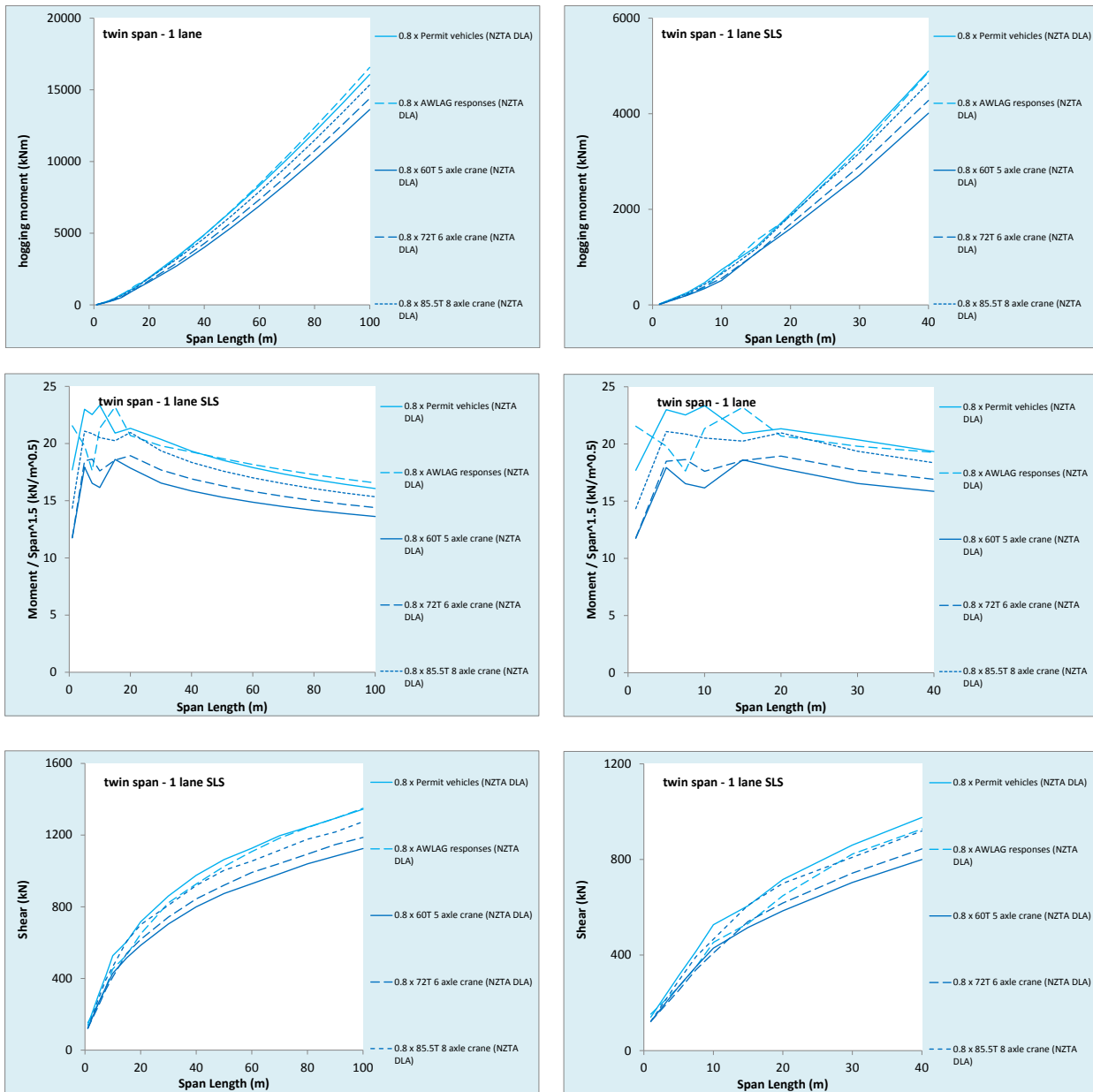
Annex D6: Envelope of structural actions from all vehicles considered

The envelopes of the structural actions from WiM, legal vehicles, AWLAG responses, permit applications and mobile cranes were plotted. With the exception of the loading from WiM data, the coincident lane loading was applied in every case.

Graphs on the left are for spans 0 to 100m, and on the right for 0 to 40m.
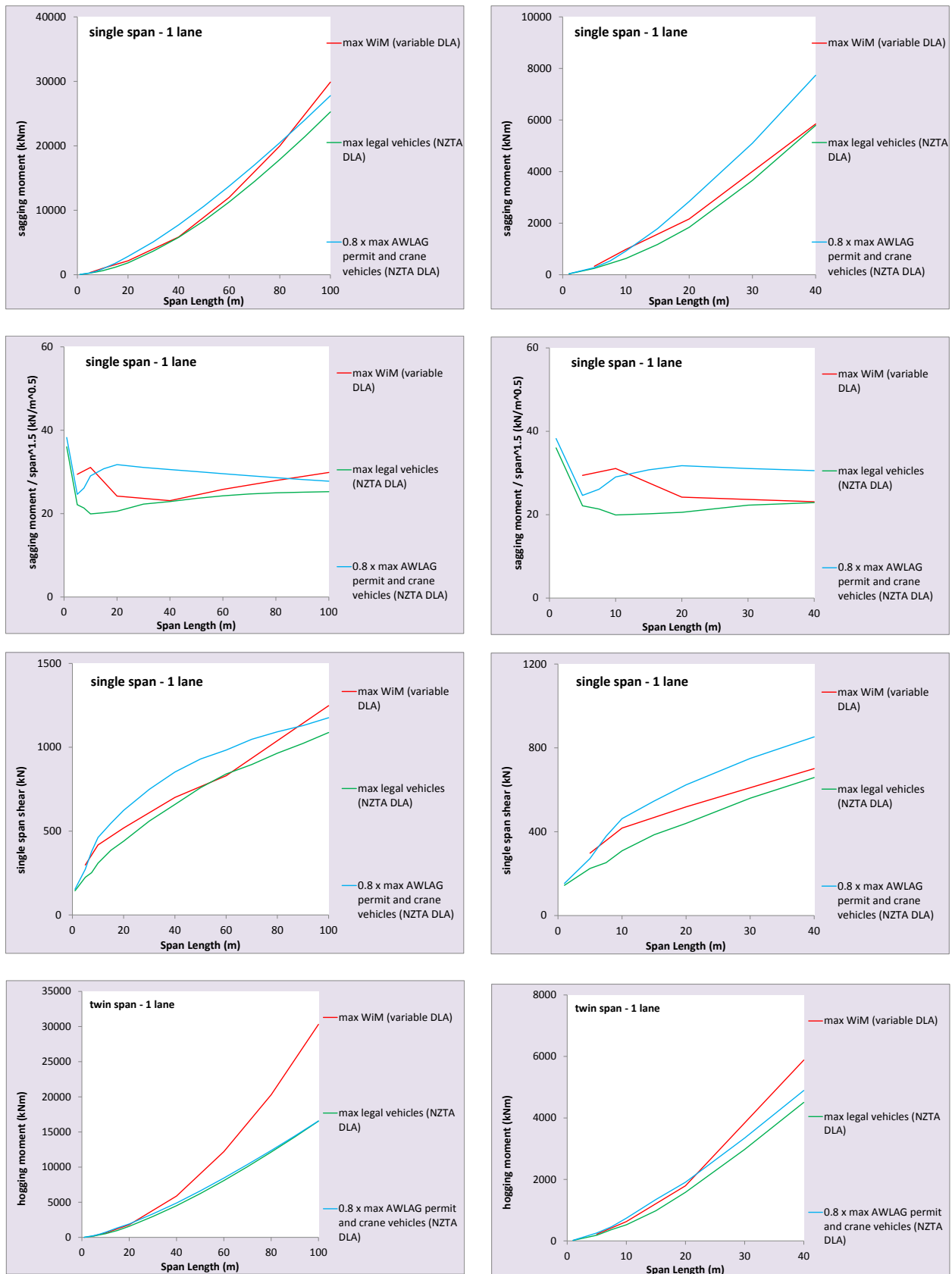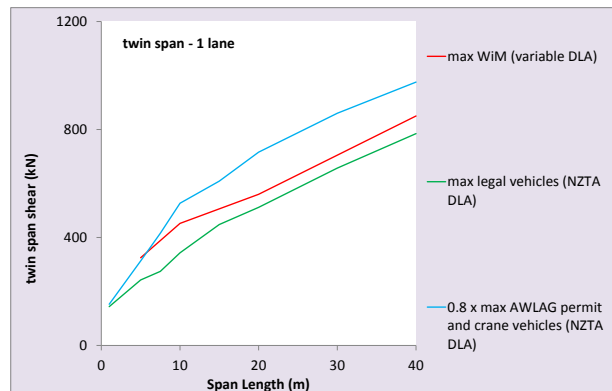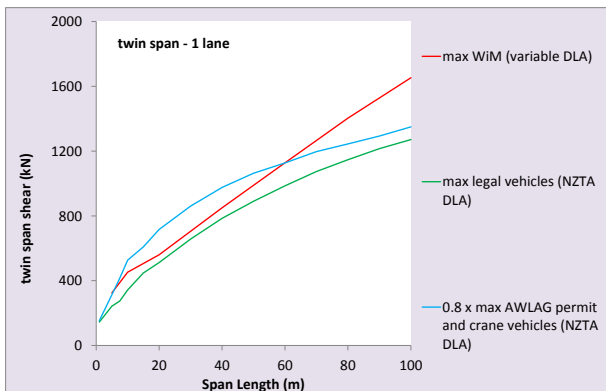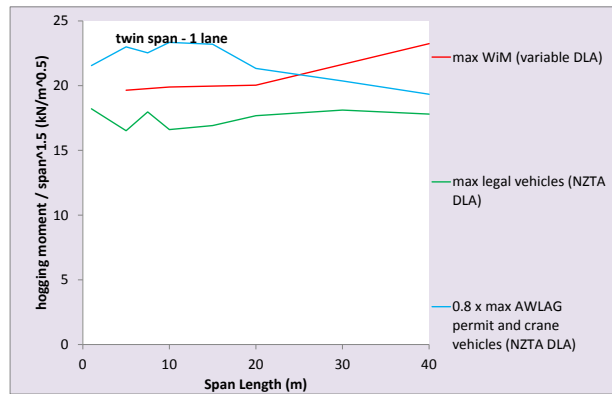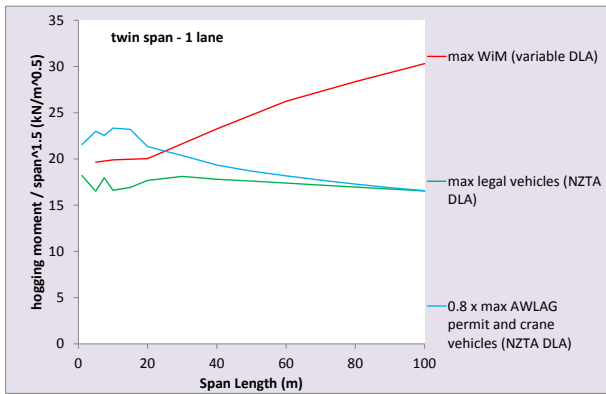
Annex D7: Upper bound structural actions and loading models

Graphs on the left are for spans 0m to 100m, and on the right for 0m to 40m.
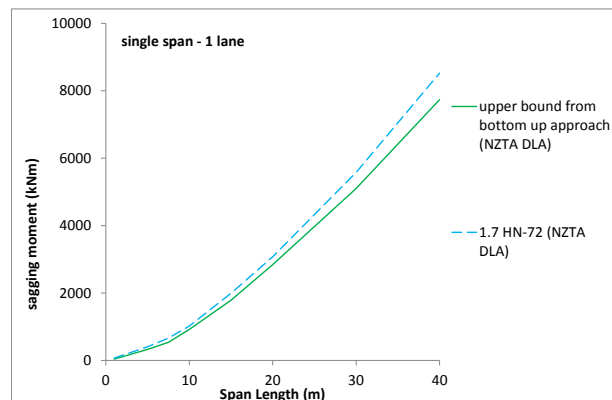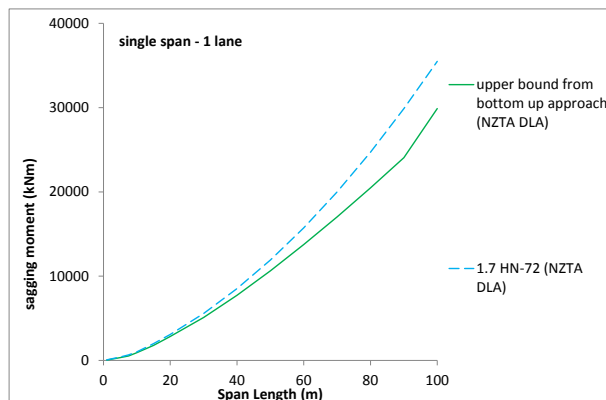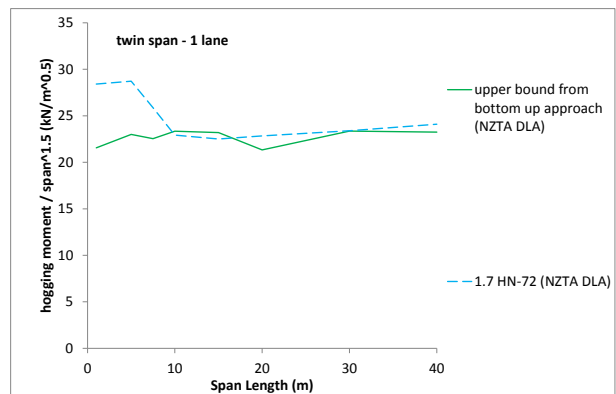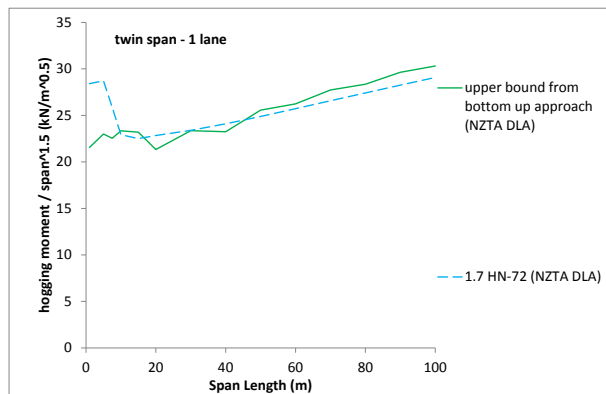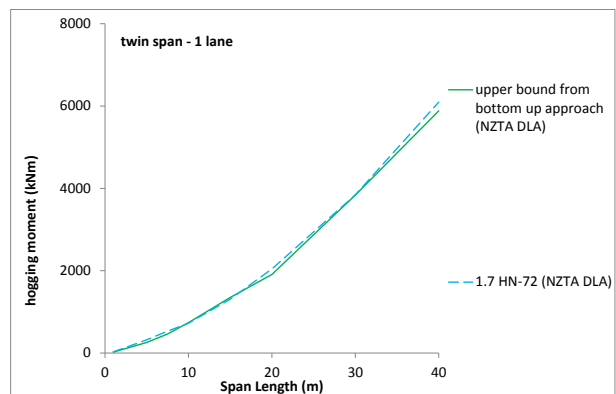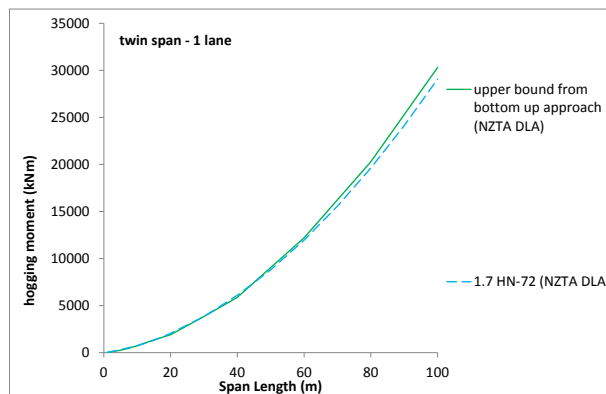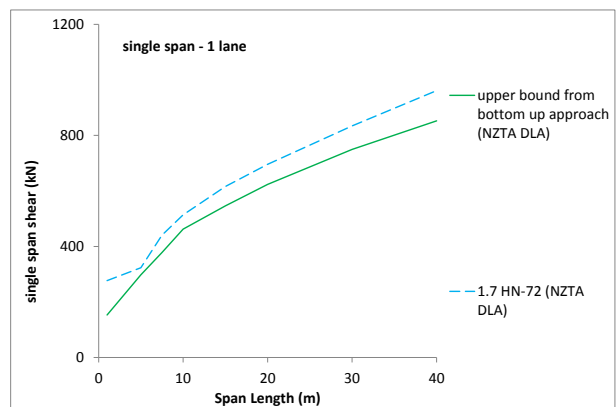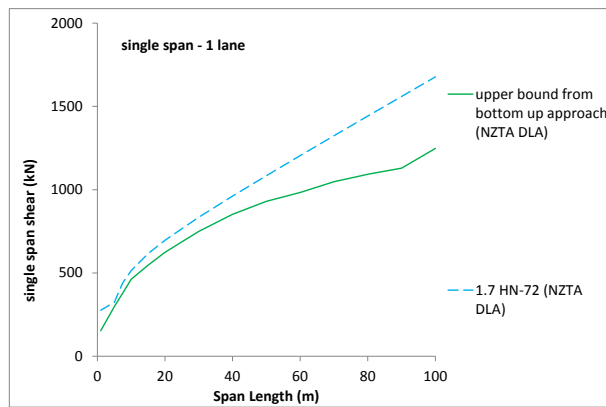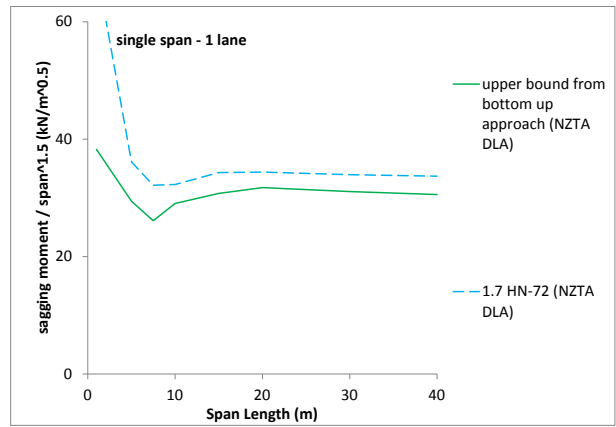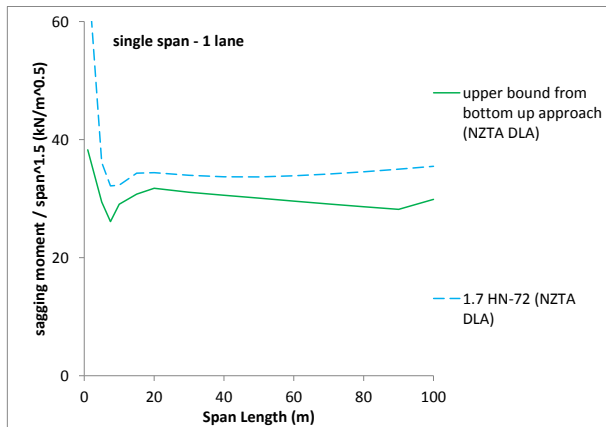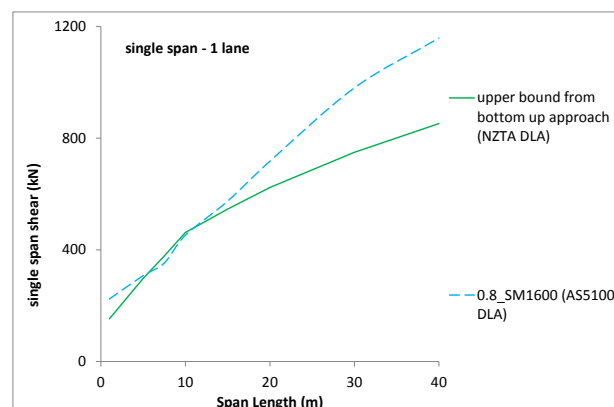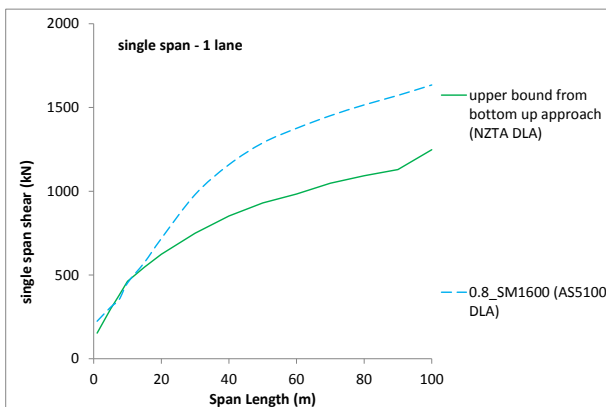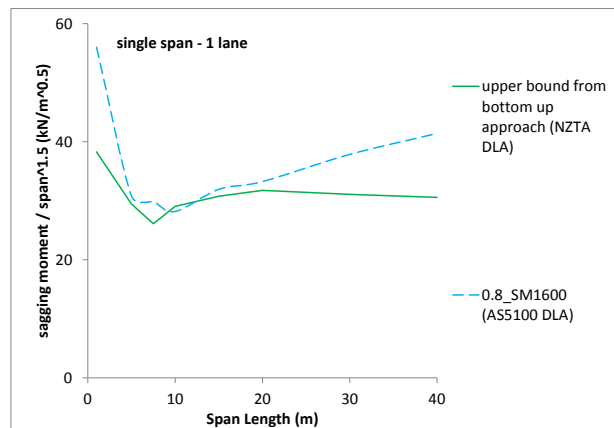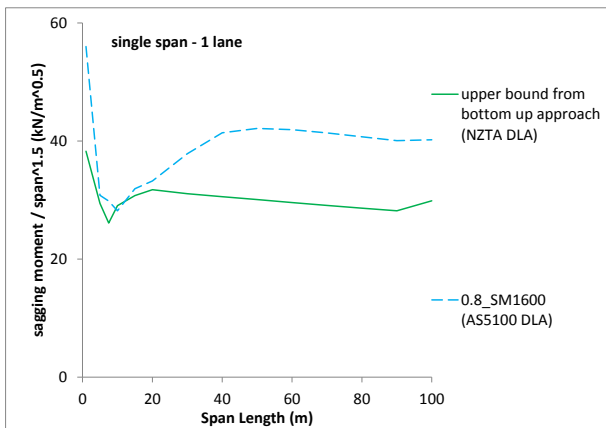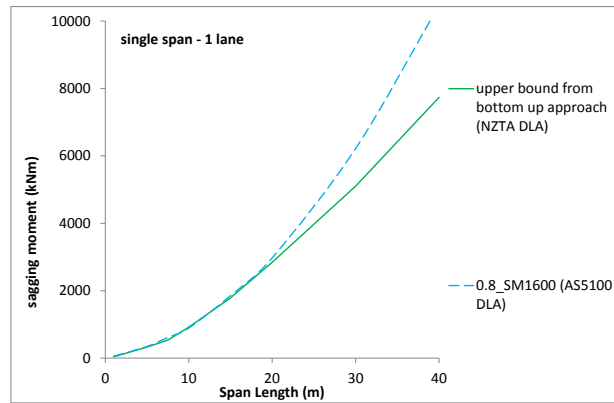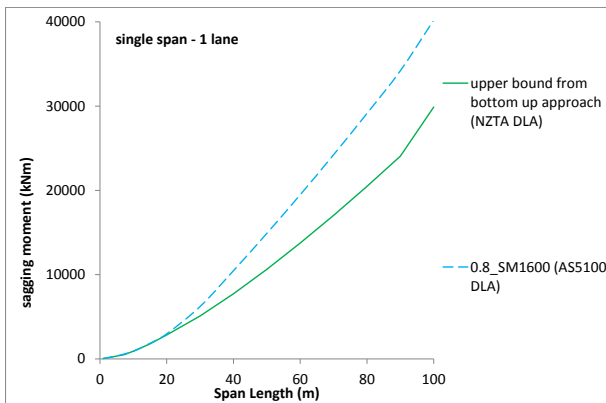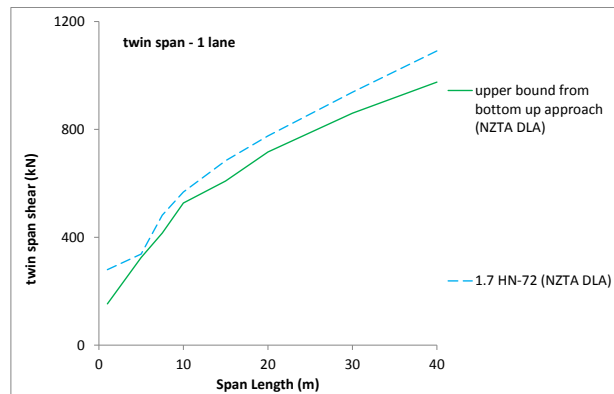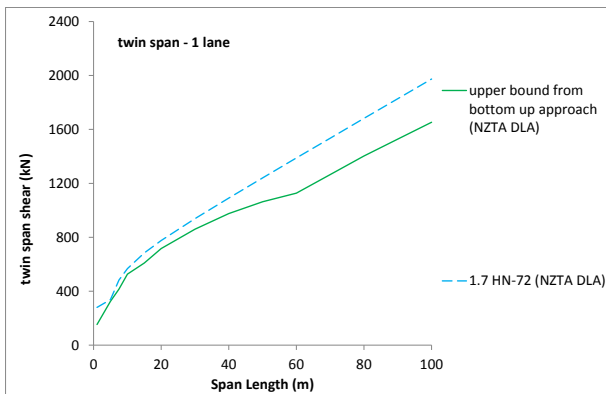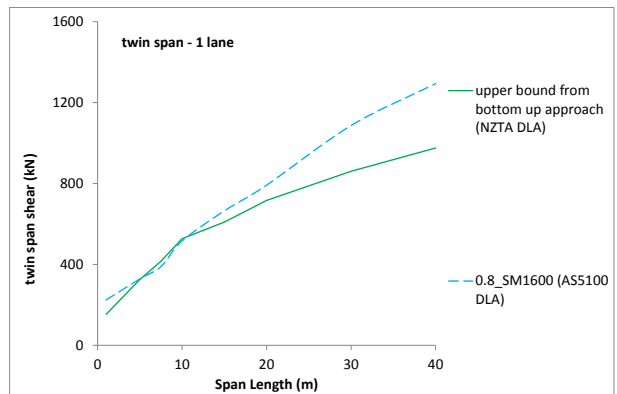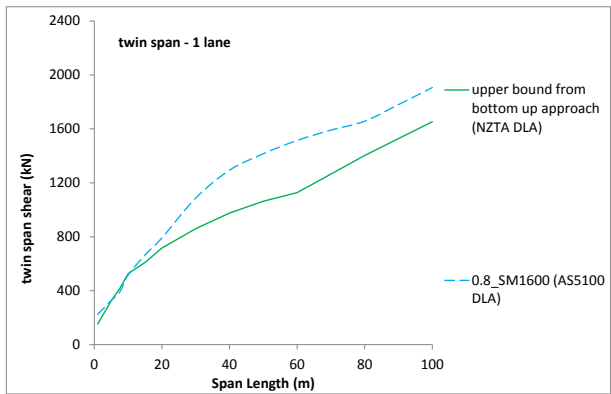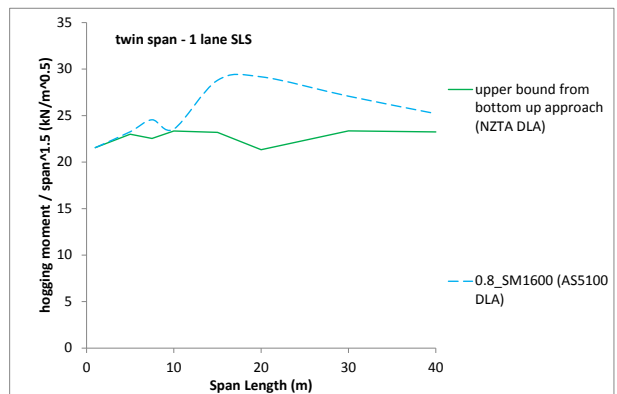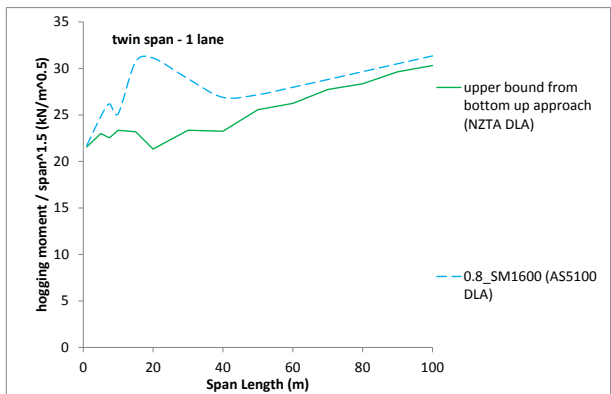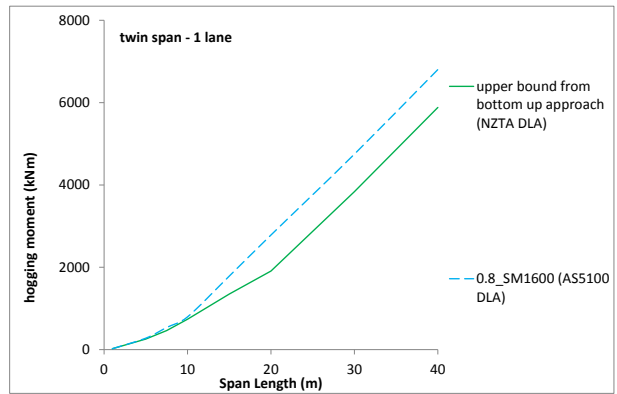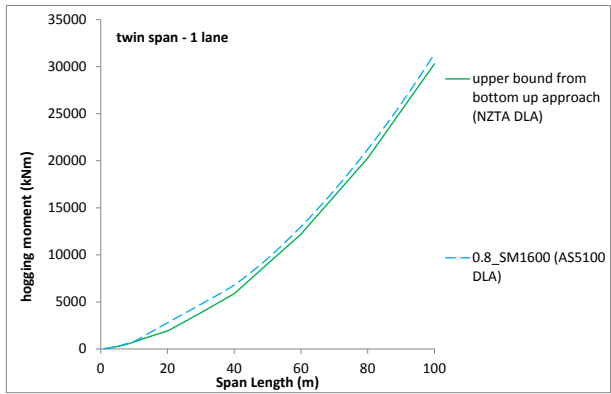
Annex D8: Comparisons between recommended and existing loading models

Graphs on the left are for spans 0m to 100m, and on the right for 0m to 40m.
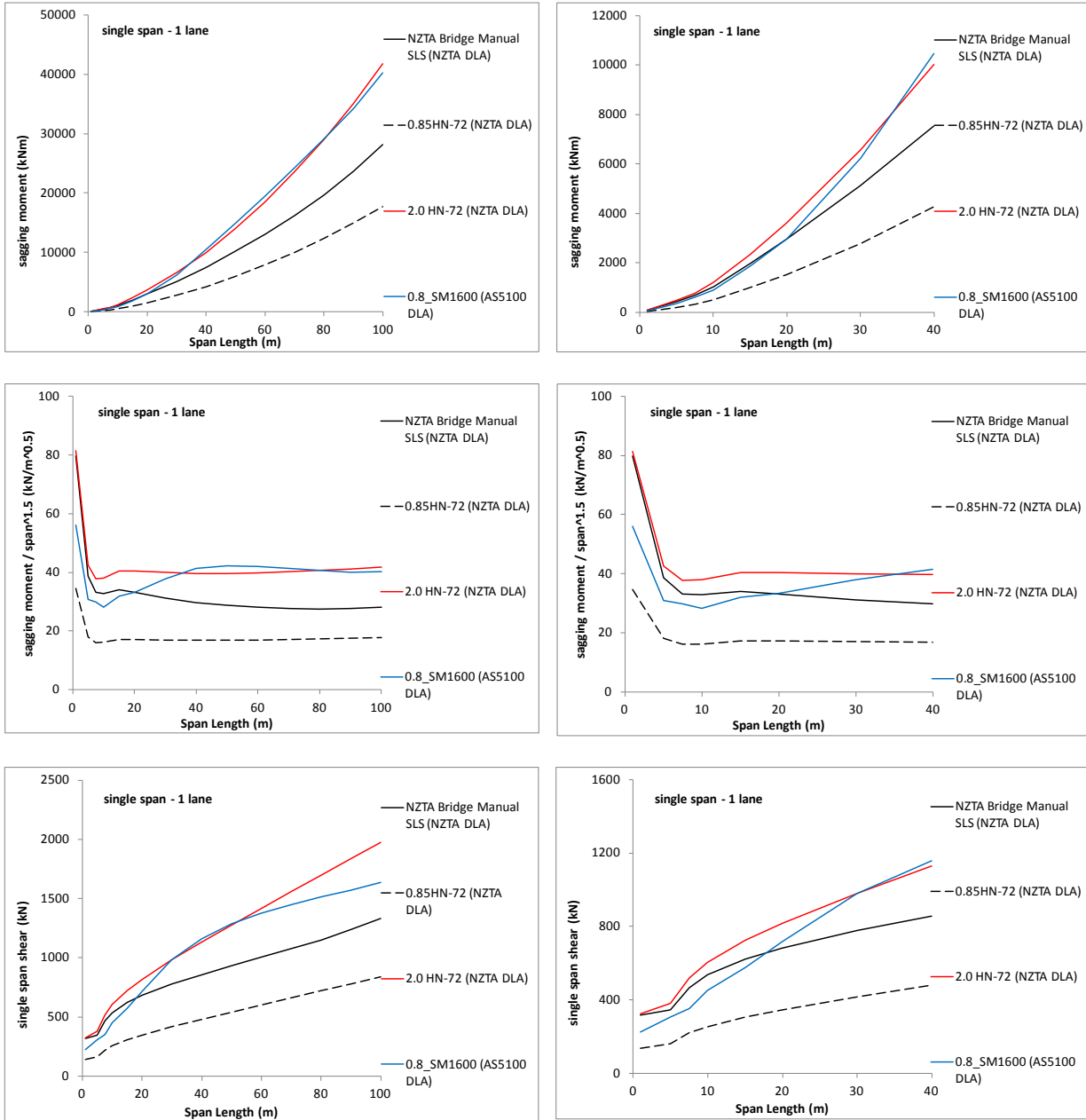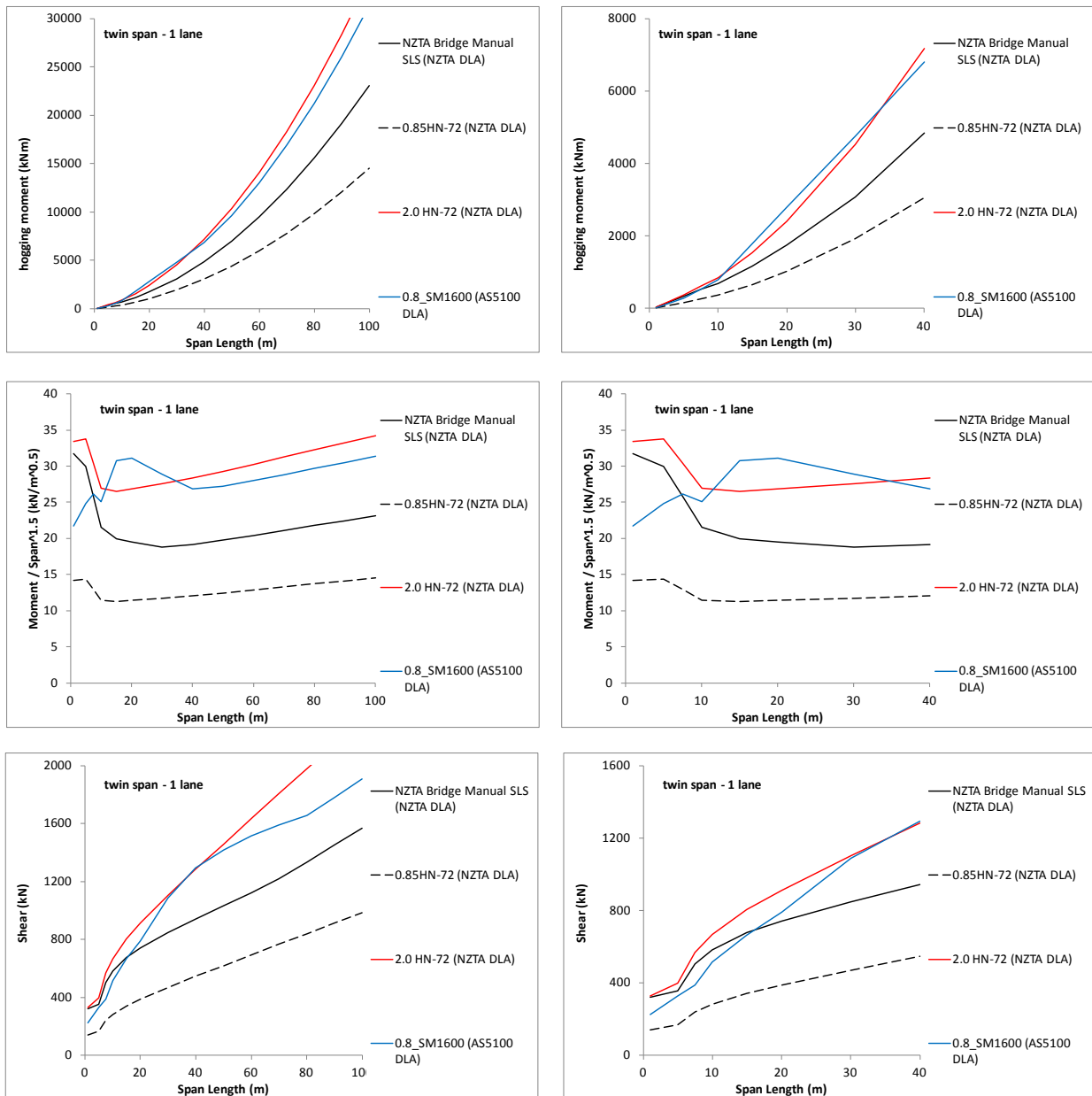
Annex D9: Review of 0.85 HN-72 evaluation loading

The design actions from 0.85 HN-72 loading are plotted against the structural actions from the NZ2222 legal vehicle, with and without coincident lane loading. The NZ2222 legal vehicle is a truck and trailer combination that complies with the Class 1 mass and dimension requirements. The vehicle has a gross combination mass of 44t.

Graphs on the left are for spans 0m to 100m, and on the right for 0m to 40m.
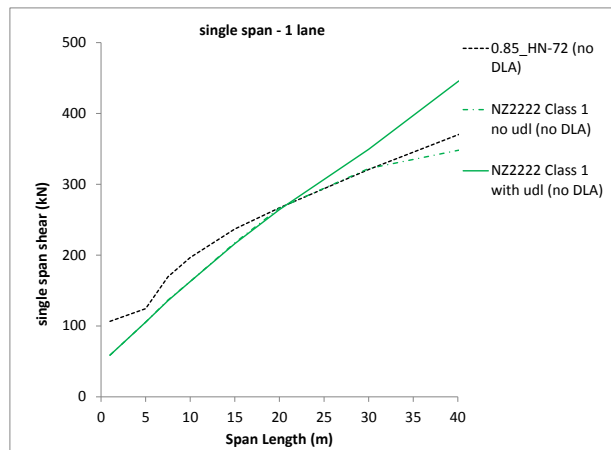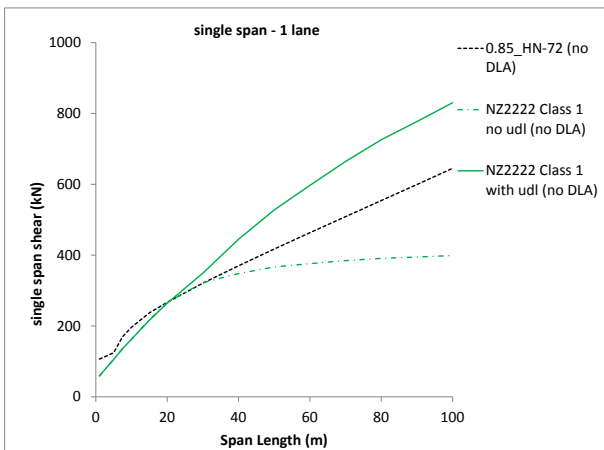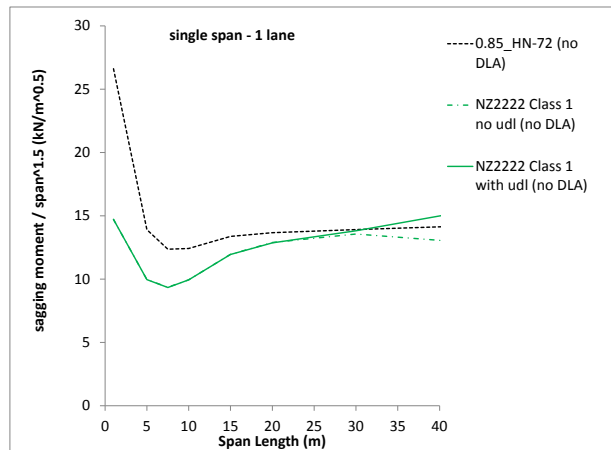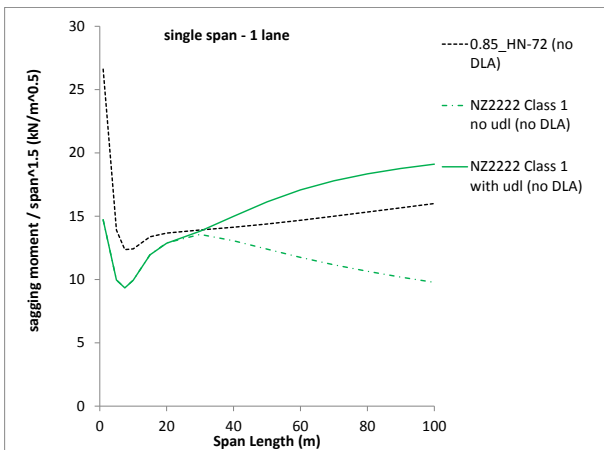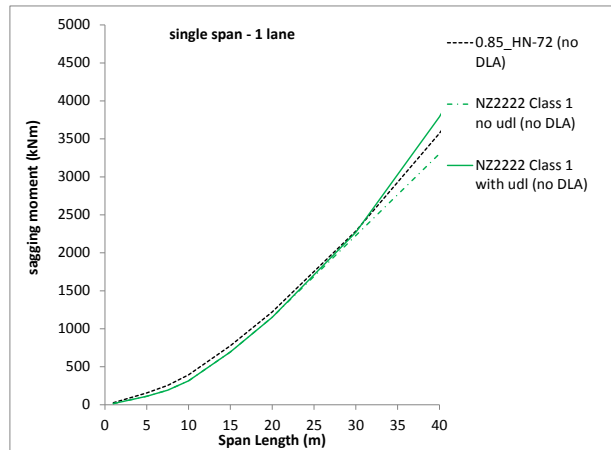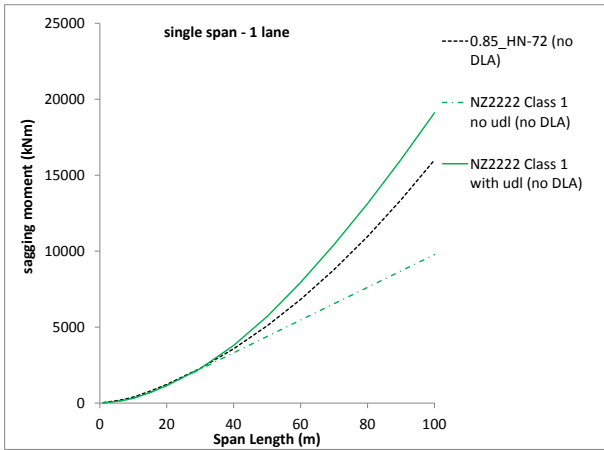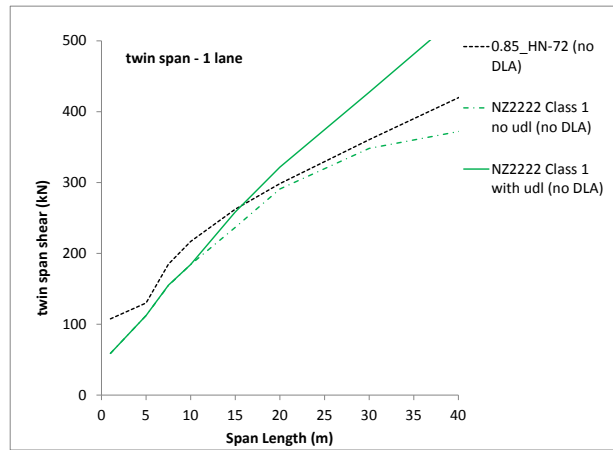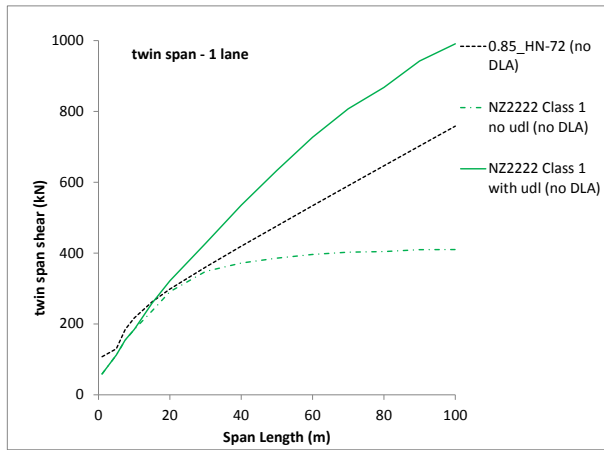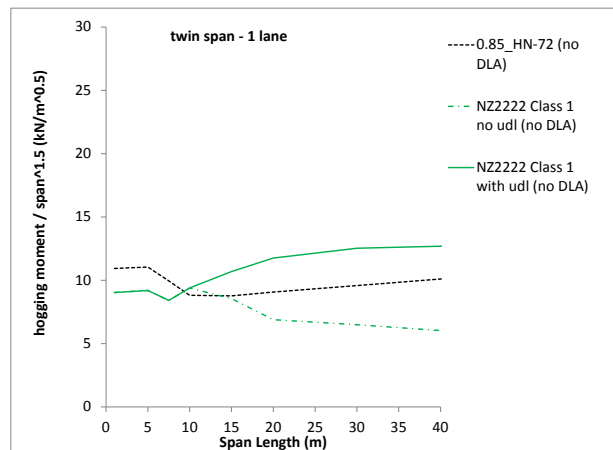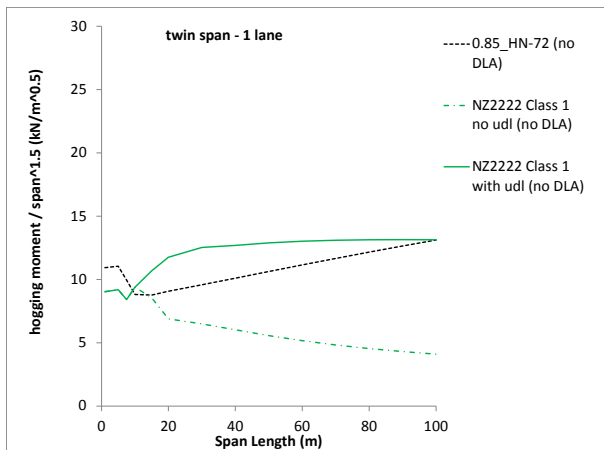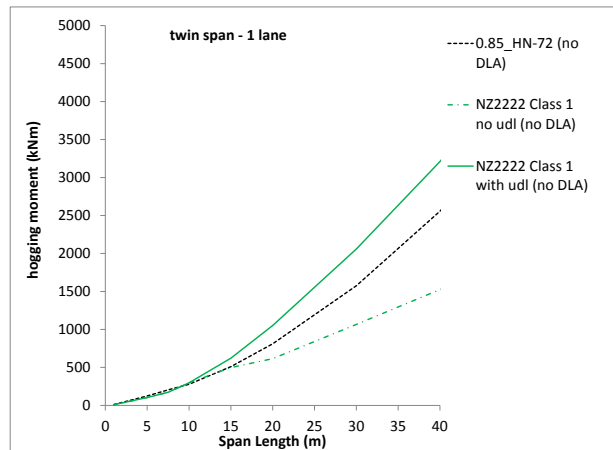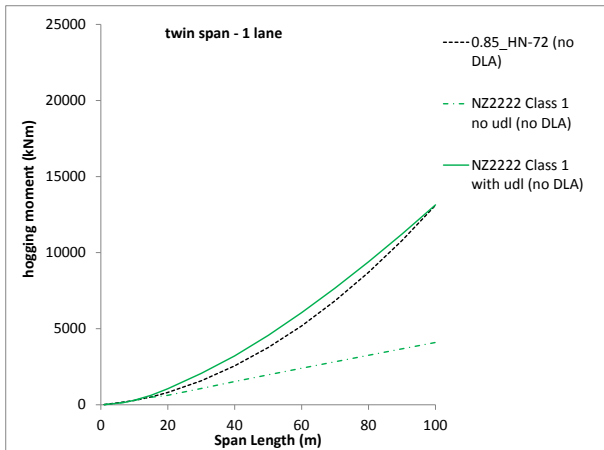
Annex D10: Proposed evaluation loading

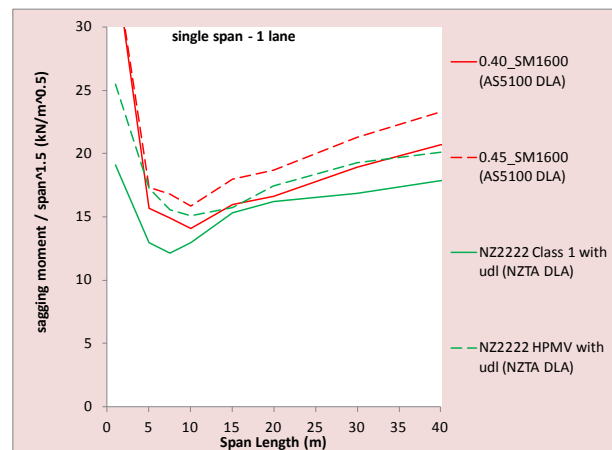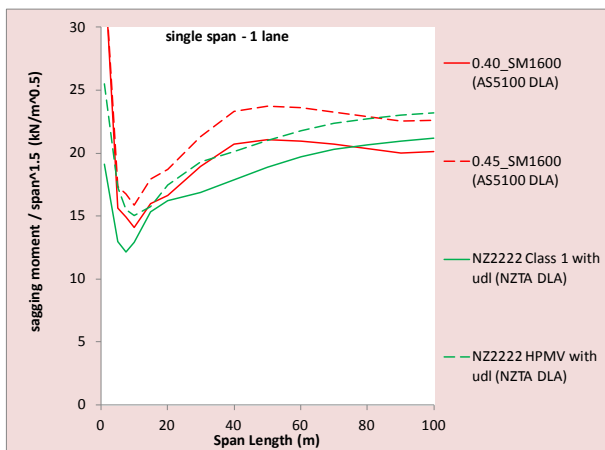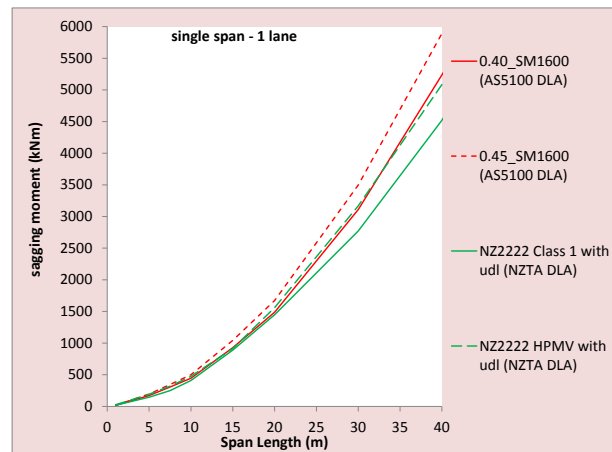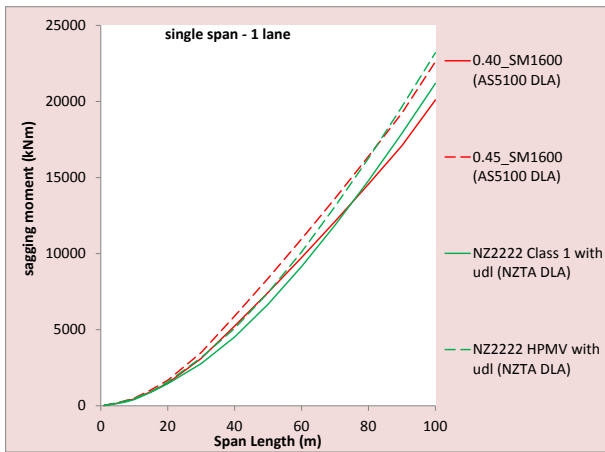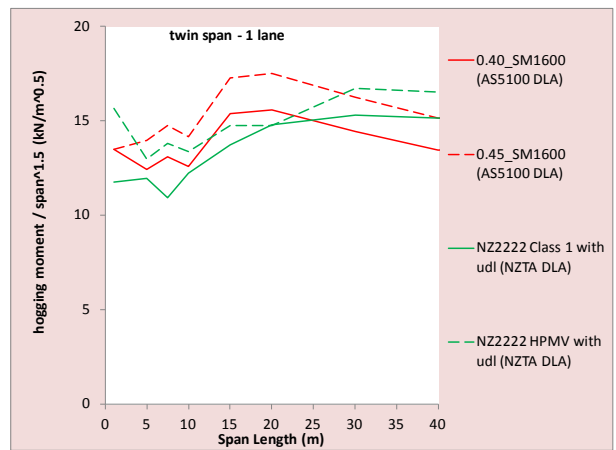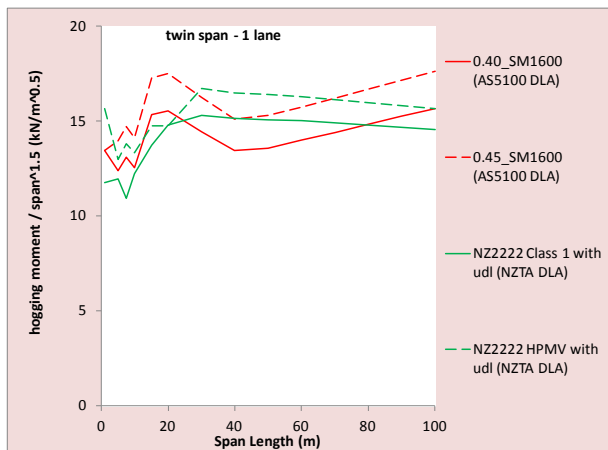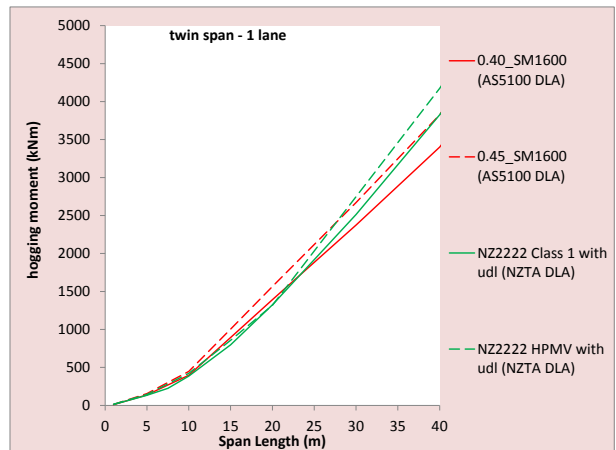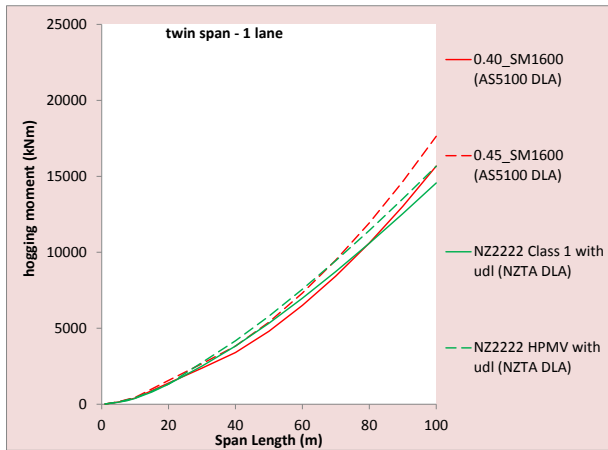The proposed evaluation loading based on the *Bridge manual* HN-72 loading (first 12 graphs) and the AS5100 SM1600 loading (second 12 graphs) are plotted against the loading from legal vehicles with coincident lane loading.

Graphs on the left are for spans 0m to 100m, and on the right for 0m to 40m.
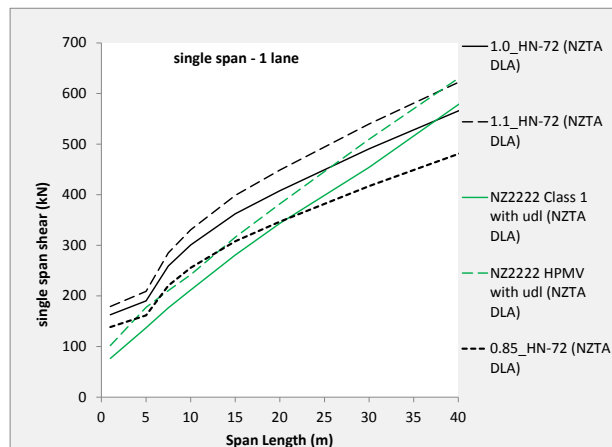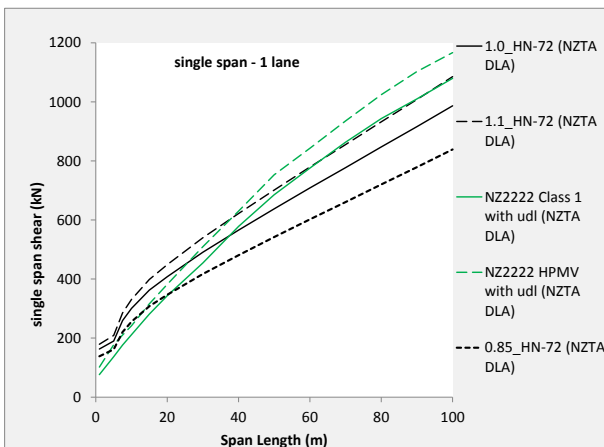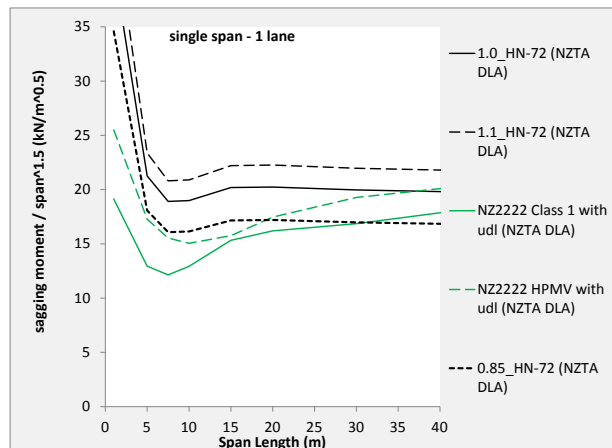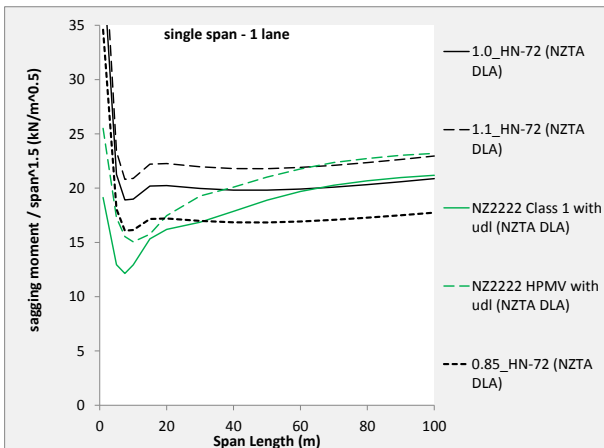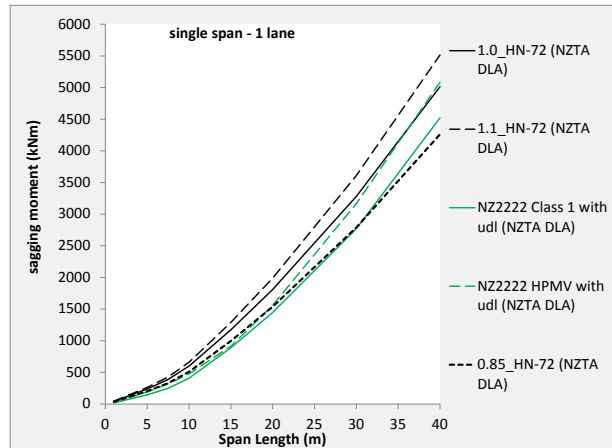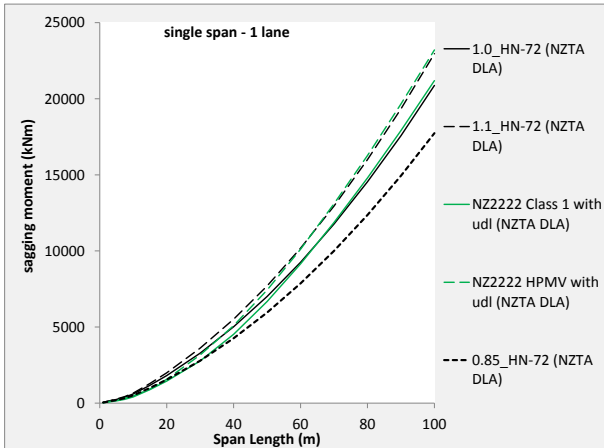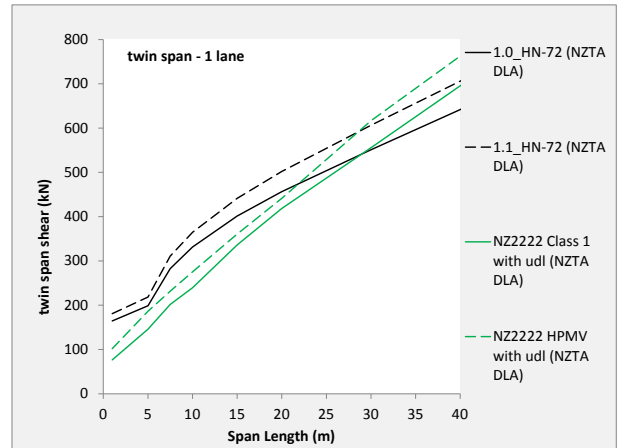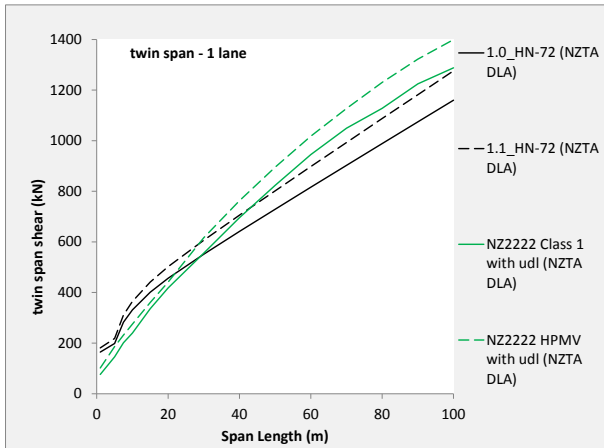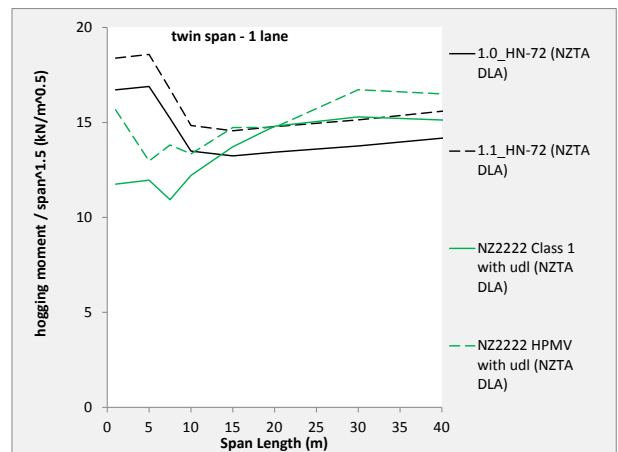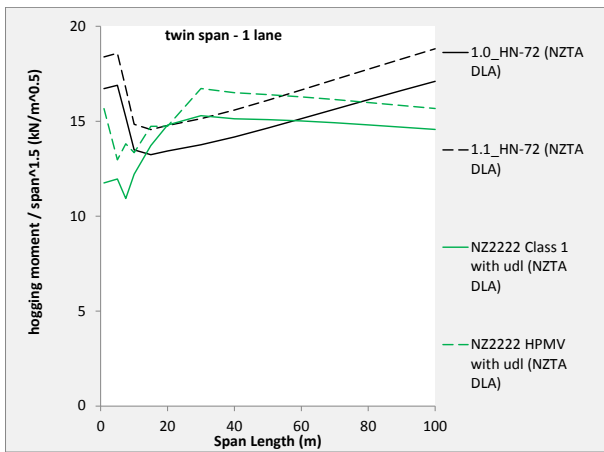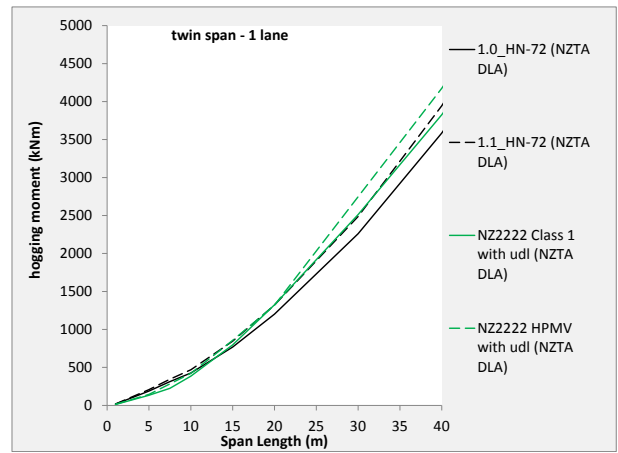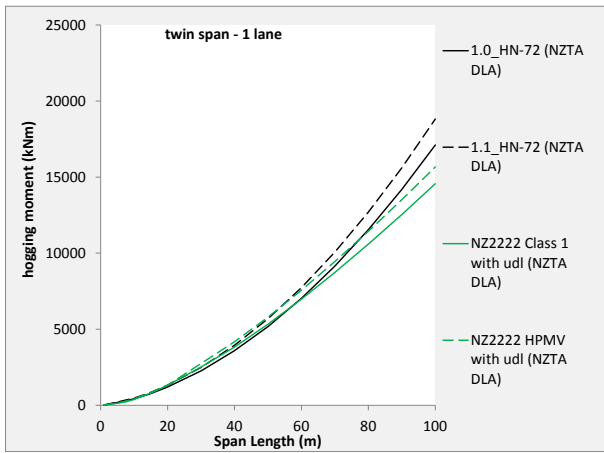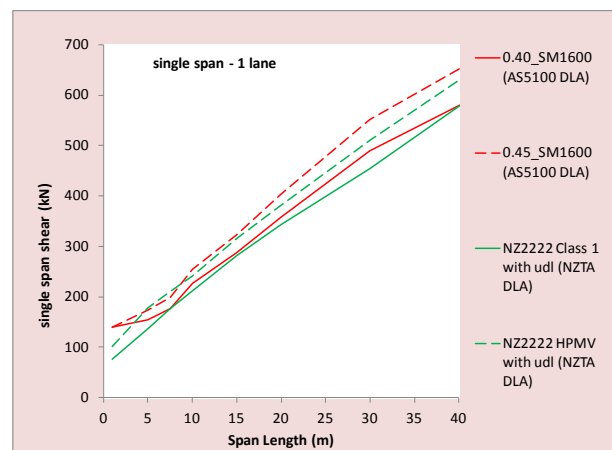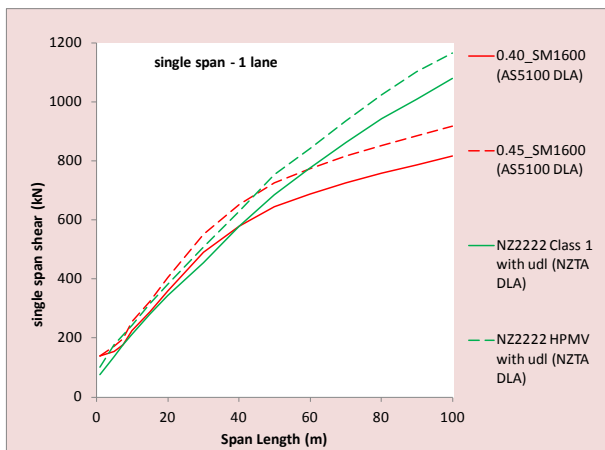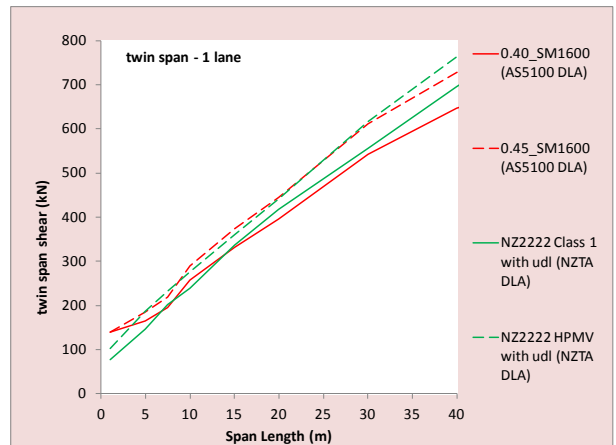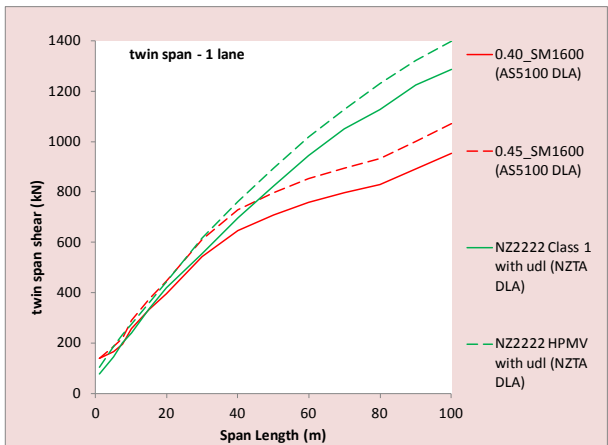
Appendix E: Stakeholder consultation

E1   Introduction

As part of the research and following a consultation meeting of the research team with members of the Axle Weights and Loadings Advisory Group (AWLAG) on 24 May 2012, a brief questionnaire was distributed to industry via the members of the AWLAG committee on 8 June 2012. The date for return was set as 20 July 2012. This appendix provides a summary of the responses received. A copy of the consultation questionnaire is included in section E4.

The first two questions asked the transport company to rate certain issues out of 5, with 5 being a serious concern/issue and 1 being an insignificant issue or concern. The remaining questions either asked for opinions/comments and/or information on the size and type of their fleet. There were 15 responses received by the due date, which were collated and the results and averages of the responses included for each question below, with a brief comment accordingly.

E2   Survey responses

Question 1: What problems does your company experience in vehicle movements?
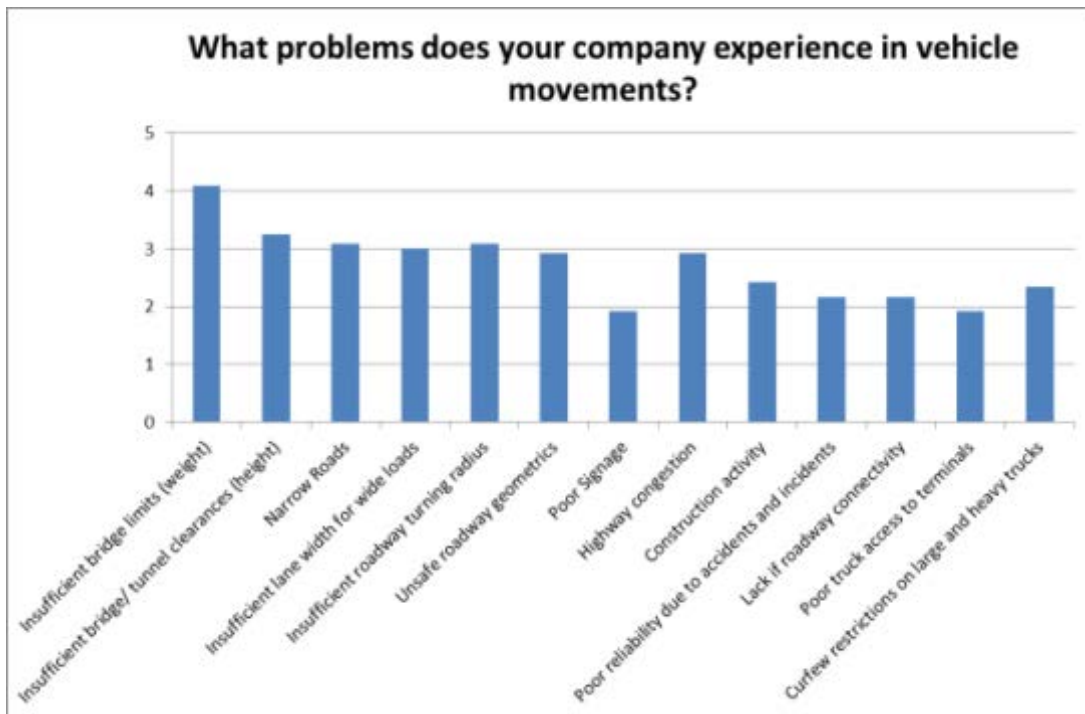
From the responses to the survey, it appears that insufficient weight limits on bridges is the biggest issue for the surveyed companies. This issue was ranked at least four out of five for all but one company. Insufficient bridge and tunnel clearances are the second highest rated issue, suggesting that constraints due to bridges are the most significant issue. Only poor signage and poor truck access to terminals scored an average below two out of five.

Question 2: How much impact does the problem create for the company?
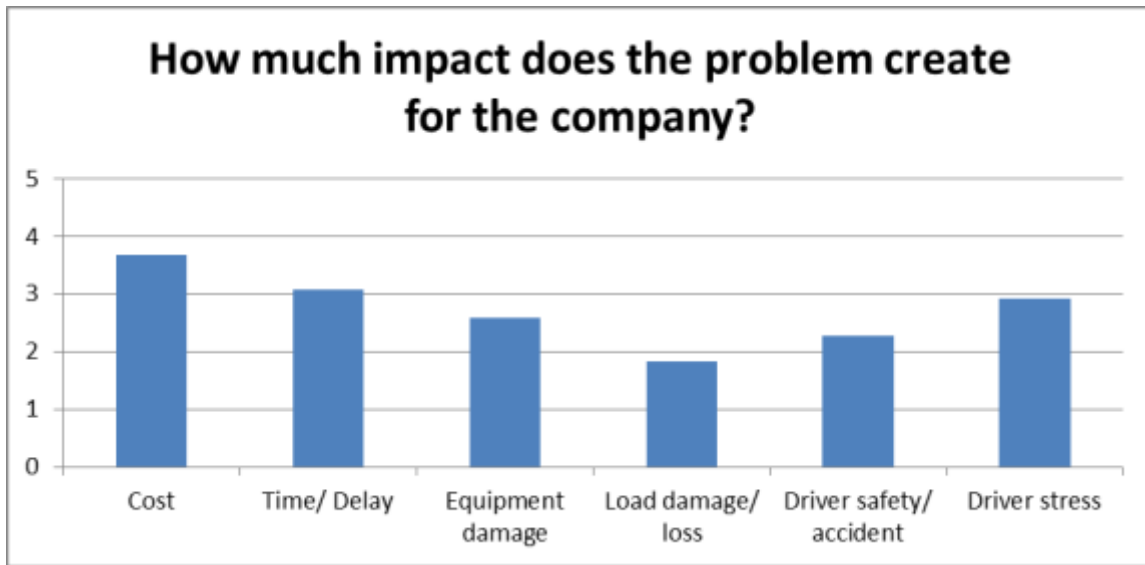
Cost is the biggest impact on the surveyed companies as a result of the issues described in question 1. This is followed by time/ delay and driver stress.

Question 3: What is the company able to do to compensate for the problems?

There are a range of mitigation measures used to limit the impact of the issues faced, these include:

*   doing route surveys

*   using experienced drivers on difficult routes

*   using alternative routes

*   load scheduling.

Question 4: Do the problems faced limit the type of vehicle you use?
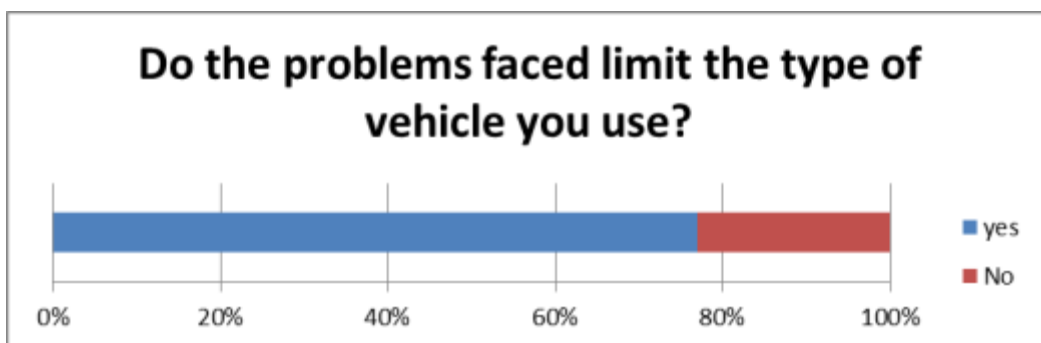
73% of respondents indicated that the problems they face places limitations on the vehicles that they use.

Comments on this question generally indicated that the companies were unable to adopt new technology from Europe or introduce HPMV vehicles.

Question 5: What is the company's level of use of vehicles?

The companies surveyed covered all types of vehicles: single unit truck/ vehicle, vehicle and trailer, tractor with 1 trailer and tractor with 2+ trailers.

Question 6: What is the company's fleet size?

Companies surveyed hold fleets of between five vehicles up to over 250.

Question 7: What type of vehicles would you like to be able to use the public highway in the future?

These ranged from 52t distributed over 20.7m and 8 axles to mobile all terrain cranes with 145.2t distributed over 16.5m and 11 axles. The detail responses are included in section E3.

E3 Responses to question 7

Question 7 asked what type of vehicles the respondent would like to be able to use on the public highway in the future? The following responses were received:

Heb Structures

Vehicle description:		
Wheelbase (metres):		
	axle weight (tonnes)	axle spacing (metres)
front axle	7.0	4.318
axle 2	9.0	1.375
axle 3	9.0	3.03
axle 4	12.0	1.24
axle 5	13.0	6.5
axle 6	15.0	2.4
axle 7	15.0	2.4
axle 8	15.0	2.4.
axle 9	15.0	
axle 10		
axle 11		

Waikato Crane Services

	axle weight (tonnes)	axle spacing (metres)
Vehicle description:	Mobile All Terrain Cranes	
Wheelbase (metres):		
front axle	13.2	1650
axle 2	13.2	1650
axle 3	13.2	1650
axle 4	13.2	1650
axle 5	13.2	1650
axle 6	13.2	1650
axle 7	13.2	1650
axle 8	13.2	1650
axle 9	13.2	1650
axle 10	13.2	1650
axle 11	13.2	

Auckland Cranes Ltd

	axle weight (tonnes)	axle spacing (metres)
Vehicle description:	Mobile All Terrain Cranes	
Wheelbase (metres):		
front axle	13.2	1650
axle 2	13.2	1650
axle 3	13.2	1650
axle 4	13.2	1650
axle 5	13.2	1650
axle 6	13.2	1650
axle 7	13.2	1650
axle 8	13.2	1650
axle 9	13.2	1650
axle 10	13.2	1650
axle 11	13.2	

Unidentified 1

	axle weight (tonnes)		axle spacing (metres)
Vehicle description:	Low Bed, Type. Semi-Trailer Unit		
Wheelbase (metres):			
front axle	6.50	Single Tyre	
axle 2	9.00	Twin Tyre	4.50
axle 3	9.00	Twin Tyre	1.40
axle 4	15.00	Row of 8	6.50
axle 5	15.00	Row of 8	2.40
axle 6	15.00	Row of 8	2.40
axle 7			
axle 8			
axle 9			
axle 10			
axle 11			

Emmerson Transport

	axle weight (tonnes)	axle spacing (metres)
Vehicle description:	Truck and Trailer 24 metres	
Wheelbase (metres):		
front axle	5	1.88
axle 2	5	4.00
axle 3	7.5	1.32
axle 4	7.5	4.80
axle 5	6.75	1.20
axle 6	6.75	6.18
axle 7	6.75	1.32
axle 8	6.75	
axle 9		
axle 10		
axle 11		

TNL Group

	axle weight (tonnes)	axle spacing (metres)
Vehicle description:		
Wheelbase (metres):		
front axle	6	4m
axle 2	7.5	1.3
axle 3	7.5	3.4
axle 4	7	1.5
axle 5	7	1.5
axle 6	7	6m
axle 7	7	1.5
axle 8	7	1.5
axle 9	7	
axle 10		
axle 11		

McIntosh Brothers

	axle weight (tonnes)	axle spacing (metres)
Vehicle description:	GMK5100 Mobile Crane	
Wheelbase (metres):	5·8	
front axle	11.35	2.54
axle 2	11.0	1.65
axle 3	11.5	2.00
axle 4	11.35	1.65
axle 5	11.40	
axle 6		
axle 7		
axle 8		
axle 9		
axle 10		
axle 11		

L&A Cotton Heavy Haulage Ltd

Vehicle description:		
Wheelbase (metres):		
	axle weight (tonnes)	**axle spacing (metres)**
front axle	9.0	
axle 2	9.0	4.25
axle 3	9.0	1.33
axle 4	12.5	2.92
axle 5	12.5	1.30
axle 6	14.15	6.47
axle 7	14.15	2.40
axle 8	14.15	2.40
axle 9	14.15	2.40
axle 10		
axle 11		

E4   Copy of the questionnaire

Freight Industry Survey

Company (freight operator):

Date:

New Zealand Transport Agency has commissioned Asset Management Research Topic TAR 11/04 *A New Vehicle Loading Standard for Road Bridges in New Zealand*. The primary purpose of this research is to determine a new vehicle loading standard for the design and evaluation of road bridges and other highway infrastructure in New Zealand.

We are conducting a short survey in order to determine the impact that road/highway infrastructure has on your current operations, and to ask what possible vehicles you may wish to use in the future.

1.   What problems does the company experience in vehicle movements?

Condition	Rate from 1 to 5 1 -not a problem 5 -very serious problem
Insufficient bridge limits (weight)	1 2 3 4 5
Insufficient bridge/tunnel clearances (height)	1 2 3 4 5
Narrow roads	1 2 3 4 5
Insufficient lane width for wide loads	1 2 3 4 5
Insufficient roadway turning radius	1 2 3 4 5
Unsafe roadway geometrics	1 2 3 4 5

Poor Signage	1 2 3 4 5
Highway congestion	1 2 3 4 5
Construction activity	1 2 3 4 5
Poor reliability due to accidents and incidents	1 2 3 4 5
Lack of roadway connectivity	1 2 3 4 5
Poor truck access to terminals	1 2 3 4 5
Curfew restrictions on large and heavy trucks	1 2 3 4 5
Other (specify) _____	1 2 3 4 5

_____

2. How much impact does the problem create for the company?

Impact	**Rate from 1 to 5**
	1 -not an impact
	5 -very serious impact
Cost	1 2 3 4 5
Time/Delay	1 2 3 4 5
Equipment damage	1 2 3 4 5
Load damage/loss	1 2 3 4 5
Driver safety/accident	1 2 3 4 5
Driver stress	1 2 3 4 5
Other (specify) _____	1 2 3 4 5

_____

3. What is the company able to do to compensate for the problems?

_____
_____
_____
_____
_____
_____

4. Do the problems faced limit the type of vehicle you use?

☐ No

☐ Yes

Comment_____
_____
_____
_____
_____

5. What is the company's level of use of vehicles?

Single unit truck/vehicle _____

Vehicle & trailer _____

Tractor with 1 trailer _____

Tractor with 2+ trailers _____

6. What is the company's fleet size?

☐ One vehicle

☐ 2-5 vehicles

☐ 6 – 10 vehicles

☐ 11 -25 vehicles

☐ Greater than 25 vehicles – Approx Number:_____

7. What Vehicles would you like to be able to use the Public Highway in the Future?

The NZ Transport Agency wants to consider what provision should be made for heavier vehicles that operators may wish to use in the future. So that we can make an appropriate provision for future vehicles we are interested to know what vehicles (if any) you would like to use on the road network, but are currently unable to use because of load restrictions. Any information that you can provide will be taken into consideration in the vehicle loading standard that this research recommends.

Possible examples might include large mobile cranes, certain passenger buses, bulk tankers, some agricultural machinery etc.

To use this information we require you to provide the axle weights and axle spacings for any vehicles that you want considered. The sample table below illustrates the minimum information that we require.

Vehicle description:		
Wheelbase (metres):		
	axle weight (tonnes)	**axle spacing (metres)**
front axle		
axle 2		
axle 3		
axle 4		
axle 5		
axle 6		
axle 7		
axle 8		

axle 9		
axle 10		
axle 11		

Example of the minimum information required:

Vehicle description:		6 axle "European" crane
Wheelbase:		2.55m
	axle weight (tonnes)	axle spacing (metres)
front axle	12	1.85
axle 2	12	2.85
axle 3	12	1.65
axle 4	12	2.58
axle 5	12	1.65
axle 6	12	-

The NZ Transport Agency and its Researchers would like to Thank You for your contribution.

Should you have any Queries please contact:

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Appendix F: Glossary

AADTT	average annual daily truck traffic
AADT	average annual daily traffic
AASHTO	American Association of State Highway and Transportation Officials
AS5100	Australian bridge design standard
ASD	allowable stress design
ASMS	axle spacing and mass schedule
AWLAG	Axle Weights and Loadings Advisory Group
BITRE	Bureau of Infrastructure Transport and Regional Economics
Bridge manual	Published by the NZ Transport Agency. The version referred to is May 2013 unless noted otherwise.
CSA	Canadian Standards Association
DLA	dynamic load allowance (additional weight due to dynamic effects of a moving vehicle)
DMRB	UK *Design manual for roads and bridges*
EEM	NZ Transport Agency (2010) *Economic evaluation manual*, volumes 1 and 2.
GEV	generalised extreme value
GVM	gross vehicle mass
headway	as used in this report it is the distance between the trailing axle of one vehicle and the leading axle of the following vehicle
HPMV	high productivity motor vehicle
HN	traffic load configuration defined in the *Bridge manual*
HO	traffic load configuration defined in the *Bridge manual*
LHV	longer heavier vehicle
LRFD	load and resistance factor design
M	million
MOW	Ministry of Works
National annexes	British Standards Institution (BSI) (2008) *National annex to British standard BS EN 1991-2: 2003* and British Standards Institution (BSI) (2009) *National annex to British standard BS EN 1990:2002+A1*
NCHRP	National Cooperative Highway Research Program
PBS	performance based standards
RUC	road user charges
SA	Standards Australia
SLS	serviceability limit state

SO	special order regulations (UK)
SOV	special order vehicle to simulate the vertical effects of SO vehicles
STGO	special types general order regulations (UK)
SV	special vehicle SV to simulate the vertical effects of STGO vehicles
t	tonne
TT73	proposed future truck and trailer vehicle used for the development of vehicle design loading
UDL	uniformly distributed load
ULS	ultimate limit state
VDAM 2010	Land Transport Rule: Vehicle Dimensions and Mass Amendment 2010
WiM	weigh-in-motion (recorded axle weights from moving traffic)