Screening bridges for potentially high dynamic loads using profile variance

September 2010

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NZ Transport Agency research report 416

ISBN 978-0-478-36487-3 (print)
ISBN 978-0-478-36486-6 (electronic)
ISSN 1173-3756 (print)
ISSN 1173-3764 (electronic)

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Jamieson, NJ and SM Bruce (2010) Screening bridges for potentially high dynamic loads using profile variance. *NZ Transport Agency research report 416*. 68pp.

Keywords: abutments, acceleration approaches, displacement, dynamic load, profile variance, roughness.

An important note for the reader

Acknowledgements

We would like to thank those who have acted as steering group members and peer reviewers for this project, including Mr Rudolph Kotze and Mr Nigel Lloyd of the NZ Transport Agency, Mr John Reynolds of Opus International Consultants and Mr Howard Chapman. Their help and advice has been much appreciated.

Abbreviations and acronyms

HCV:	heavy commercial vehicle
IRI:	International Roughness Index
NAASRA:	National Association of Australian State Road Authorities
NZTA:	New Zealand Transport Agency
RAMM:	road asset maintenance management
RP:	route position
SH:	State Highway
UK:	United Kingdom

Contents

Executive summary

This study was undertaken in 2008–2009 to assess the potential to use road profile variance as a screening tool to (a) identify bridges where poor approaches or abutments may be contributing to high dynamic bridge loading, and (b) to identify where maintenance may be required to reduce impact loading on bridges. Profile variance is a measure of the difference between the actual road profile and its moving average over selected moving average lengths (short, medium and long). The potential advantage of using profile variance is that this data, which is calculated from the raw road profiles, is currently available from the NZ Transport Agency's (NZTA) Road Asset Maintenance Management (RAMM) database. This would help to avoid the need for time-consuming and expensive bridge monitoring.

The research was based on a comparison of measured dynamic bridge deflections on selected bridges on the state highway (SH) network against the road profile variance, roughness and geometry data from the RAMM database.

Sections of SH1 and SH2 were selected from the state highway network as having a range of bridge sizes and types that were representative of New Zealand's state highway bridge stocks. Roughness (using profile variance and standard National Association of Australian State Road Authorities (NAASRA) measurements) and geometry (gradient curvature and crossfall) data were extracted from RAMM for the selected road sections. Video records from the annual state highway survey and aerial photos were obtained for each bridge on the selected road sections, including the immediate approaches.

An on-road test programme was used to measure the dynamic deflections of the first span of selected bridges on the chosen road sections. These bridges were chosen by a steering group of industry representatives and network consultants, and included bridges with low roughness and profile variance, as well as bridges with high roughness and profile variance. The first span at both ends of the selected bridges was instrumented with an accelerometer to measure the vertical motion of the span under the freestream traffic loading. This freestream traffic included heavy commercial vehicles (HCVs), which are expected to generate the greatest dynamic loads. An instrumented test vehicle was also driven over the bridges and their approaches to measure the vehicle's response to the road roughness elements associated with the different elements, ie the approaches, abutments, bridge deck and joints. One of the bridges selected (Mangatewai-iti Bridge) was found to have had smoothing works carried out immediately prior to the on-road testing.

The bridge acceleration records were doubly integrated to provide time histories of the deflections associated with the freestream traffic passing over each bridge. These were compared with the instrumented vehicle response data, and with the profile variance and roughness data to assess the effects of different levels and wavelengths of profile variance on the dynamic response of bridges.

Conclusions on the current state of the profile variance and roughness of bridges and approaches, and the effects of that profile variance on dynamic bridge loading are described below, followed by recommendations for further work.

Conclusions

The conclusions from this study of the effects of profile variance on the dynamic response of bridges are as follows:

Profile variance of bridges and approaches – current status

Comparison of the RAMM data for a wide range of bridges on the state highway network showed that for the bridges and the approaches within 100m of the bridge abutments:

- NAASRA roughness on bridges and bridge approaches is, on average, around 35% higher than on the adjacent roads, up from around 60 NAASRA counts/km to around 80
- short wavelength (3m) profile variance on bridges and bridge approaches is, on average, around 60% higher than on the adjacent roads, up from around 1.2mm^2 to around 2mm^2
- medium wavelength (10m) profile variance on bridges and bridge approaches is, on average, around 100% higher than on the adjacent roads, up from around 7mm^2 to around 14mm^2
- long wavelength (30m) profile variance on bridges and bridge approaches is, on average, around 90% higher than on the adjacent roads, up from around 135mm^2 to around 260mm^2
- maximum values of short, medium and long wavelength profile variance on bridges and approaches often exceed the thresholds for poor ride quality used by the UK Roads Board. These maximum values on bridges and their approaches most often occur around the bridge abutments and bridge deck joints.

Vehicle driveover accelerations

Measurements of an instrumented vehicle's response to the approaches, abutments and decks of selected bridges showed the following:

- The vertical accelerations were highest in the immediate vicinity of the bridge abutments, although peaks were also observed around the bridge deck joints.
- Maximum peak-to-peak vertical accelerations on the first bridge spans ranged from around 4.4m/s^2 to 13.2m/s^2.

Bridge deflections

Measurements of the dynamic deflections of the first bridge spans caused by HCVs in the freestream traffic showed that maximum peak-to-peak dynamic deflections on the first bridge spans ranged from around 0.7mm to 5.6mm for HCVs. Average peak-to-peak dynamic deflections on the first bridge spans ranged from around 0.6mm to 2.6mm.

Comparison of bridge deflections and profile variance

Comparison of the measured dynamic bridge deflections of the first bridge spans and the measured values of profile variance for short, medium and long wavelengths showed the following:

- The dynamic bridge deflections, both the maximum and average peak-to-peak values, generally increased with increasing profile variance across the short (3m), medium (10m) and long (30m) wavelengths.

- The most significant predictor of the dynamic bridge displacements, and hence the dynamic loads, was the short (3m) wavelength profile variance, with a correlation coefficient of around 70%, even allowing for expected variations in traffic loads and speeds.
- Differences in the 3m profile variance between each end of the same bridge showed consistent differences in the dynamic displacements.
- The 3m wavelength profile variance can be used to rank bridge approaches and abutments for dynamic loads and ride quality. Using the UK Roads Board threshold of $10mm^2$ for poor truck ride quality, the 3m profile variance can also potentially be used to screen bridges for high dynamic loading and to identify bridges where maintenance or smoothing works could potentially reduce these loads and improve bridge life.

Recommendations

The recommended research actions arising from this study of the effects of profile variance on the dynamic response of bridges are as follows:

- Carry out a national screening of bridges to identify bridges with 3m wavelength profile variance levels over $10mm^2$ that are potentially exposed to high dynamic loads, and encourage regional and network bridge consultants to assess smoothing options.
- Extend the bridge displacement measurements to cover a wider range of bridge types, sizes, profile variance values and traffic loadings.
- Determine whether the smoothing works on the Mangatewai-iti bridge approaches reduced the peak 3m profile variance values by comparing the 2008/09 network survey values with those from the 2009/10 survey.
- Select one or two bridges with peak 3m profile variance values on the approaches/abutments in excess of the UK Roads Board $10mm^2$ threshold for poor ride quality as candidates for smoothing work.
- Measure the first span static and dynamic displacements on these bridges using an HCV, varying both its load and speed up to the maximum acceptable limits, including the proposed new weight limits.
- Carry out smoothing works on the bridges.
- Repeat the dynamic displacement measurements using the same combinations of vehicles, loads and speeds.
- Identify the changes in the profile variance levels resulting from the smoothing works.
- Quantify the relative effects of changes in 3m profile variance on the dynamic displacements and dynamic impact factors, including variation with vehicle speed and load.

Abstract

Profile variance, which is already listed in NZTA's RAMM database, is a measure of the difference between the actual road profile and its moving average over selected moving average lengths. The relationships between measured dynamic bridge deflections and road profile variance and roughness values derived from raw road profiles for short, long and medium wavelengths were examined during 2008–2009. These relationships between profile variance and bridge response were used to investigate the potential use of profile variance as a predictor of dynamic vehicle loads and as a means of screening New Zealand bridges for mitigation of damage, and for targeted and proactive maintenance.

1. Introduction

1.1 Aims

The objective of this research project, undertaken in 2008–2009, was to assess whether the short, medium and long wavelength profile variance data stored in the Road Asset Maintenance Management (RAMM) database could be used as a screening tool to (a) identify bridges where poor approach or abutment conditions may contribute to high dynamic loading, and (b) identify where maintenance or improvements may be indicated to reduce impact loading on bridges.

This research will be relevant to management of the New Zealand state highway bridge stocks by the NZ Transport Agency (NZTA) and their local network managers, particularly given the increasing numbers of heavy commercial vehicles (HCVs) and the likely increase in maximum allowable axle weights. The most significant advantage of using profile variance data is being able to better target maintenance and improvements to bridges, without necessarily having to resort to extensive and costly full-scale measurements.

1.2 Background

When bridges are constructed or maintained, the general intention is to make them fit as seamlessly as possible into the road network. However, this is not always achieved because while roughness may be managed, the actual road profile is not normally targeted. Differences will often appear between the desired smooth road surface and what is built. The resulting undulations in the surface profile or 'road roughness' can be described by their amplitude or by their wavelength. Typically, for pavement wavelengths of around 0.3m or less, the effects of any undulations in the road profile are absorbed by a vehicle's tyres. At most normal road travel speeds, wavelengths of 50m or more do not have any significant effect on vehicle response. It is therefore generally accepted that the important range of wavelengths lies between ~0.5 and 50m.

These road surface undulations can occur on or near bridges in a number of ways. At longer distances from the bridges (~50m or more), the road surface undulations are mostly caused by the original road construction and by pavement changes that have occurred with time and traffic, eg settling, maintenance and resurfacing. The immediate approaches to bridges suffer from similar issues relating to roughness, but are often more prone to settlement of fill materials behind the abutments. The roughness undulations on the bridge decks themselves are primarily related to the flexible bridge spans, the span joints and the deck surfacing.

The effects of undulations in the road surface can be transferred or magnified through a vehicle's tyres and suspension in such a way that it affects on the vehicle, the load and its driver; the reverse is also true, particularly for HCVs. The effects of the vehicle's mass bouncing on the tyres and suspension can be transferred back to the road surface, thus causing pavement loading due not only to the mass of the vehicle (static loads) but also to the dynamic movement (dynamic loads). This load transfer is of particular significance on bridges and bridge approaches, as this has potential for high levels of roughness to cause heavy trucks to 'bounce' onto bridges, thereby possibly creating high dynamic loads on the bridges, particularly on the first bridge spans. These high dynamic loads resulting from poor bridge approaches and abutments are known to be a problem area that can exacerbate impact damage as trucks pass onto the bridge deck. As measuring dynamic or static loads is an expensive and time-consuming process, engineers, managers and researchers need a tool that can use existing routinely collected data to screen bridges for potentially high dynamic loads and to identify where maintenance can be best targeted to reduce these loads. Furthermore, the NZTA is also tasked with providing good ride quality on roads and bridges, as well as providing good quality bridge asset management. The ability to identify poor ride quality issues and less than desirable dynamic loading on bridges, even if the overall impact on structural performance is minor, is also important.

1.3 Need for research

Currently, the effects of HCV traffic on bridges can only be established through visual inspections and full-scale measurements of either static or dynamic bridge loads, both of which can be time-consuming and expensive. Accordingly, significant benefits may be gained by being able to assess whether bridges are exposed to high dynamic loads, and thereby potential damage or fatigue, through the use of data that is currently collected on a routine basis.

It is known that the road roughness has a significant effect on the loads that vehicles put on the underlying pavement, leading to deterioration and damage. This has led to the development of roughness progression and pavement deterioration models that are used to predict pavement life. Bridges also suffer from the effects generated by the pavement roughness. However, the roughness measures that have been used in the past, and which have been routinely collected and included in NZTA's RAMM database (using both National Association of Australian State Road Authorities units (NAASRA) and the International Roughness Index (IRI)), do not provide any information on the wavelength characteristics of the surface, particularly the long wavelengths that affect the ride characteristics of HCVs more severely or the short wavelengths that can significantly affect dynamic impact loading. It was not known whether the impact loads created by heavy trucks as they drive onto bridges are generated by the medium and long wavelength roughness sometimes associated with the approaches, or by the short wavelength roughness that is mostly associated with any seal changes on the immediate approaches or the abutments themselves.

The profile variance method allows the wavelength content to be determined. It quantifies the level of variation of the road profile from its moving average over different lengths corresponding to different wavelengths. It has been routinely used in the United Kingdom (UK) for a number of years, where wavelengths of 3m, 10m and 30m have been used (UK Roads Board 2003). High levels of 3m variance typically arise from short wavelength features such as seal changes, bridge abutments, faulting, potholes and poor reinstatements. Extremely high levels of 3m variance may be linked with the presence of severe wheelpath cracking. The 10m variance is often influenced by short undulations, possibly arising from different rates of pavement settlement, or localised subsidence of reinstatements and subsurface utilities. High levels of 30m variance will have more influence on the users' perception of ride quality, particularly that of truck drivers.

The NZTA has been processing the road profile information to generate 3m, 10m and 30m profile variance data since 2004. Accordingly, it is possible to assess whether profile variance can be used to identify bridges that may be prone to high dynamic loading because of roughness issues, and whether this can be used to target maintenance funding.

1.4 Methodology

The primary goal of the research was to assess whether routinely collected profile variance data could help target maintenance funding, provide focus for more detailed inspections and perhaps lessen the need for full-scale testing for bridges.

The research programme actions were to:

- generate a database comprising of the profile variance, geometry and roughness data for selected sections of the state highway (SH) network that contain a wide range of bridge types, approach conditions and traffic levels
- extract video records from the RAMM database for each of the bridges within the selection road sections
- carry out visual inspections of bridges on the selected sections
- make a final selection of bridges for full-scale testing
- carry out 'driveover' tests with an instrumented vehicle to measure the response data on the chosen bridges and their approaches
- measure the vertical accelerations that are induced by HCVs in the first span at both ends of each bridge
- investigate the relationships between the measured driveover vehicle response, the measured bridge accelerations, derived bridge deflections, and the road geometry and profile variance data
- assess the use of profile variance levels as a screening tool for identifying bridges that may be prone to high dynamic impact loads.

1.5 Scope of the report

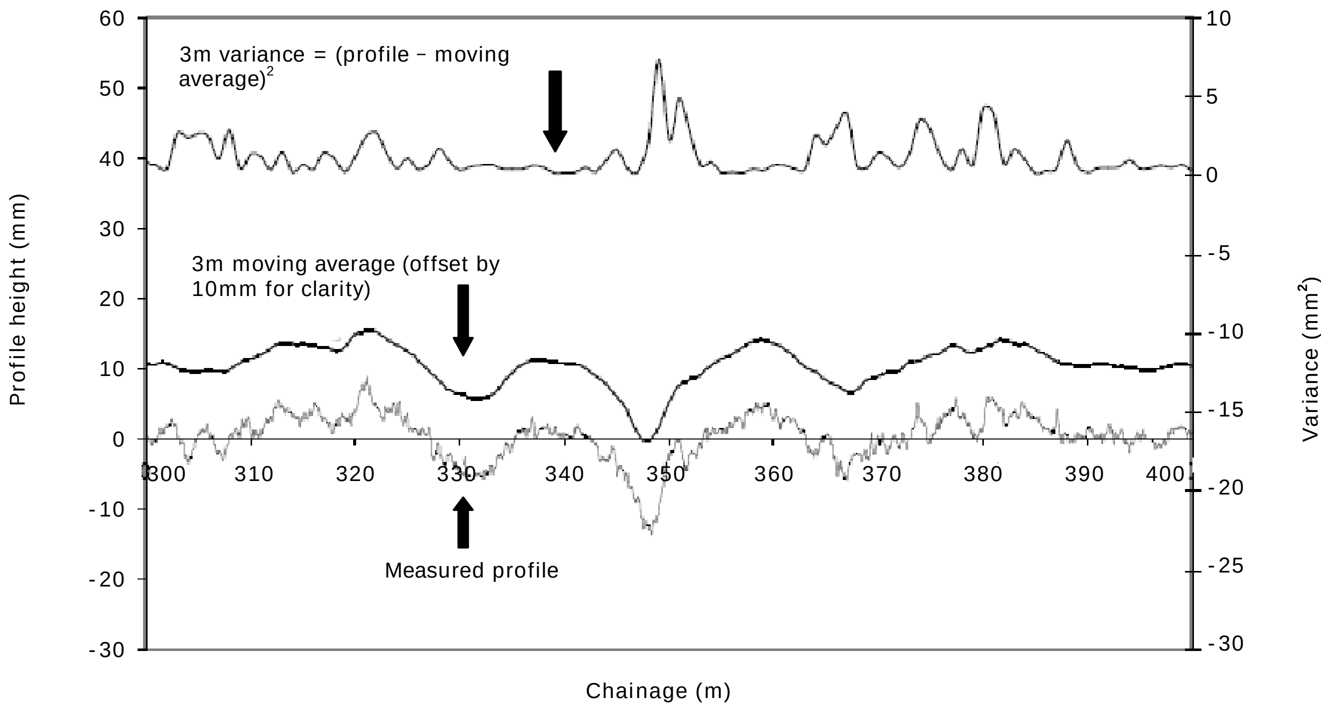
This report presents the results of a study comparing measured vehicle and bridge response data with road profile variance data, and roughness and geometry data. Chapter 2 discusses road profile variance, including definitions and the threshold values used in the United Kingdom. Chapter 3 describes the generation of the project bridge/road database; extraction of roughness, profile variance and geometry data; and the selection of the bridges for the full-scale testing. In chapter 4, the vehicle driveover testing and results are discussed. The vertical bridge accelerations and results are described in chapter 5. Analysis of the full-scale measured data and the investigation of relationships with the roughness, profile variance and geometry data are covered in chapter 6. The potential use of profile variance as a screening tool is discussed in chapter 7. Finally, conclusions and recommendations drawn from the research are given in chapter 8.

2. Road profile variance

2.1 Background and definition

Profile variance is a measure of the difference between the road profile and a moving average of the road profile over selected moving average lengths. The concept of profile variance is shown graphically in figure 2.1 and the methodology for calculating profile variance is given in appendix A.

Figure 2.1 The measured profile and profile variance



In the UK, three averaging lengths are used: 3m, 10m and 30m. The road profile data is processed to compare the actual profile and the moving average of the profile over these three lengths. The results are presented in terms of the square of the difference between the moving average of the profile and the measured profile. They are reported as the 3m, 10m and 30m longitudinal profile variance, and expressed as averages over 10m lengths.

Longitudinal road profile data is recorded during the NZTA's annual survey of the state highway network. This is processed to provide a large number of variables that are currently included in the NZTA's RAMM database. Further processing of the road profile data is required to obtain information on the wavelength content of the profile, eg the profile variance.

2.2 Threshold values

In the UK, four condition categories for pavement condition in terms of profile variance have been used. These are defined by profile variance threshold criteria for each of the three averaging lengths. These condition categories, which are based on characteristic values associated with 100m pavement lengths, are:

- sound: no visible deterioration
- some deterioration: low level of concern (no action unless long lengths are affected)
- moderate deterioration: warning level of concern (investigate)
- severe deterioration: intervention level of concern (action required).

The threshold longitudinal profile variance values for urban dual carriageway and rural single carriageway roads are listed in table 2.1 for the step from moderate deterioration (investigate) to severe deterioration (action required). It is important to also note here that according to the UK Roads Board Advice Note (2003), consideration of profile variance should generally be limited to relatively straight, predominantly high-speed roads, and that variations in geometry can affect the calculation of profile variance.

Table 2.1 Profile variance threshold values for 100m lengths for urban dual carriageway and rural single carriageway roads (UK Roads Board 2003)

Wavelength (m)	Profile variance (mm^2)
3	10
10	56
30	300

In 2005, a new UK Roads Board Interim Advice Note (2005) advised that for the assessment of ride quality the moving average longitudinal profile variance was to be replaced by the enhanced longitudinal profile variance. This was intended to reduce or remove the contribution of the underlying road geometry to the variance calculations. The new methodology used in the UK uses sophisticated filters to reduce the occurrences of falsely high levels of variance that can be associated with significant changes in geometry. However, it was decided by the NZTA that New Zealand would not move to use the enhanced profile variance at this stage, primarily because this filtering removes so much of the wavelength content information from the data.

3. Bridge database generation, data extraction and site selection

3.1 Bridge database generation and data extraction

For this study, it was important that the bridges selected for the full-scale measurement programme provided the desired mix of characteristics, not only in terms of bridge sizes and types, but also with respect to levels of profile variance. Accordingly, the project steering group was asked to choose appropriate sections of the state highway network that included the desired range of bridges. The decision was made to concentrate on SH2 between Featherston and Napier, with additional sections of SH1 being included to provide additional options for particular bridge types. This provided a total of 50 bridges.

The required data, including geometry, was extracted from the RAMM database for the selected road sections. This included NAASRA and IRI roughness, profile variance and event codes that identify the locations of bridges and other items such as railway lines. Data was extracted for the entire road lengths, which covered the bridges, their approaches and the full sections between. The extraction of the data for road lengths between the bridges and their approaches was done so that the levels of roughness and profile variance on or near bridges could be compared with those representative of the rest of the network. Data relating to each of the specific bridges within the selected road sections was also extracted from NZTA's Bridge Descriptive Inventory[1] database. This data included the year of construction, the beam type, superstructure material, deck material, length and number of spans.

The data extractions provided a database with specific information on the bridges and their approaches, and a range of information on the road sections between. Additional supporting information was also gathered. Aerial photos of each bridge and approaches were obtained from Google Earth (Google Inc. 2008). Video records were extracted from the latest annual high speed state highway survey record for each bridge and its approaches in increasing and decreasing directions.

3.2 Site selection

It was intended that the steering group should make the final selection of sites for consideration for full-scale measurements. Accordingly, plots of the NAASRA roughness, profile variance (3m, 10m and 30m in each wheelpath in each direction) were prepared for each of the bridges and their approaches to complement the other bridge information. This was presented to the steering group for discussion. It was decided that the full-scale measurements should concentrate on the most common types of bridges, including a mix of types and ages, and a mix of profile variance levels both higher and lower than the UK threshold levels for poor ride quality described in table 2.1. This reduced the number of candidate sites to 20. A summary of the characteristics of these bridges is listed in table 3.1, including the maximum and minimum profile variance levels found on the bridges and the approaches within 200m of both ends.

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1 The Bridge Descriptive Inventory, also known as the Bridge Data System, is a computer database administered by the NZTA. People wanting to access this database should contact the NZTA's National Office.

Table 3.1 Candidate bridge characteristics

Feature	Bridge name									
	Tauherenikau		Waiohine		Mangateretere		Waingawa		Makakahi River (Newman)	
	Inc[d]	Dec[e]	Inc	Dec	Inc	Dec	Inc	Dec	Inc	Dec
Year	1971		2006		1962		1991		1988	
SH[a]	2		2		2		2		2	
RP[b]	905		883		883		883		825	
Disp[c]	11.4		20.3		19.2		5.77		16.3	
Cross-section of superstructure	Units with slab		Units without slab		Beam and slab, non-composite		Beam and slab composite		Beam and slab composite	
Long section of superstructure	Simple spans		Simple spans		Simple spans		Simple spans		Simple spans	
Beam type	Double hollow core units		Double hollow core units		I-beams		U-beams		I-beams	
Deck material	Reinforced concrete		Pre-stressed concrete		Reinforced concrete		Reinforced concrete		Reinforced concrete	
Superstructure material[f]	Concrete, cast in situ and reinforced		Concrete, precast and pre-tensioned		Concrete, cast in situ and reinforced		Concrete, precast and pre-tensioned		Concrete, precast and pre-tensioned	
Length	126.5		93.85		54.2		136		56.3	
Spans	10		5		3		6		3	
LWP max 3m profile variance	1.9	1.8	3.1	3.5	2.9	3.1	7.0	10.5	6.0	13.3
LWP[g] max 10m profile variance	10.8	6.8	50.4	13.2	31.7	24.7	42.3	35.3	60.9	46.7
LWP max 30m profile variance	152.3	79.5	443.1	338.0	322.7	189.0	151.5	142.2	542.1	315.4
RWP max 3m profile variance	2.2	3.1	4.7	10.2	4.0	3.8	9.8	11.3	6.3	14.4
RWP max10m profile variance	11.4	7.2	32.6	59.9	26.6	26.6	28.8	37.8	76.8	70.3
RWP max 30m profile variance	162.9	57.3	597.8	1984.1	207.5	238.7	430.2	96.7	965.8	887.7
Max NAASRA	74	94	118	138	130	122	174	198	174	196

Table 3.1 (cont.) Candidate bridge characteristics

Feature	Bridge name									
	Makakahi River (Konini)		Oruakeretaki Stream		Tamaki River		Mangatera Stream		Whakaruatapu Stream	
	Inc	Dec	Inc	Dec	Inc	Dec	Inc	Dec	Inc	Dec
Year	2002		1990		1985		1987		1927	
SH[a]	2		2		2		2		2	
RP[b]	825		772		772		772		758	
Disp[c]	0		11.2		6.91		0		7.99	
Cross-section of superstructure	Units without slab		Beam and slab composite		Beam and slab composite		Beam and slab composite		Beam and slab, non-composite	
Long section of superstructure	Simple spans		Simple spans		Simple spans		Simple spans		Simple spans	
Beam type	Double hollow core units		U-beams		I-beams		U-beams		Plate girders	
Deck material	Prestressed concrete		Reinforced concrete		Reinforced concrete		Reinforced concrete		Reinforced concrete	
Superstructure material[f]	*Concrete, precast, and pre- and post-tensioned*		Concrete, precast and pre-tensioned		Concrete, precast and pre-tensioned		Concrete, precast and pre-tensioned		Steel	
Length	74.5		55.9		42.5		51.7		58.1	
Spans	4		3		2		3		3	
LWP max 3m profile variance	7.6	0.8	11.7	13.0	13.5	14.6	13.1	6.1	12.0	9.5
LWP[g] max 10m profile variance	43.0	30.5	41.3	36.0	47.0	42.5	65.8	14.9	56.3	53.2
LWP max 30m profile variance	380.4	158.4	238.2	192.9	580.0	531.4	1571.8	458.5	398.3	442.6
RWP max 3m profile variance	13.1	1.5	11.1	11.5	12.9	16.3	7.4	3.6	5.9	9.4
RWP max10m profile variance	66.3	23.5	57.4	33.7	48.6	50.1	71.7	16.7	33.6	39.2
RWP max 30m profile variance	663.6	182.3	242.8	199.8	575.2	514.3	1566.2	501.0	236.6	335.5
Max NAASRA	181	121	157	160	224	205	163	96	200	208

Table 3.1 (cont.) Candidate bridge characteristics

Feature	Bridge name									
	Mangatewai-iti Stream		Mangatewai-nui River		Waipawa River		Karamu Creek		Ngaruroro	
	Inc[d]	Dec[e]	Inc	Dec	Inc	Dec	Inc	Dec	Inc	Dec
Year	1983		1988		1956		1929		1965	
SH[a]	2		2		2		2		2	
RP[b]	758		758		707		661		650	
Disp[c]	2.42		0.42		6.82		6.06		9.02	
Cross-section of superstructure	Beam and slab composite		Box girder		Beam and slab composite		Beam and slab composite		Beam and slab composite	
Long section of superstructure	Simple spans		Continuous spans		Simple spans		Simple spans		Simple spans	
Beam type	I-beams		Other		Plate girders		I-beams		I-beams	
Deck material	Reinforced concrete		Reinforced concrete		Reinforced concrete		Reinforced concrete		Reinforced concrete	
Superstructure material[f]	Concrete, precast and pre-tensioned		Steel		Steel		Steel		*Concrete, precast, and pre- and post-tensioned*	
Length	85.2		102.3		303.5		24.6		297.5	
Spans	4		3		14		2		12	
LWP max 3m profile variance	4.6	6.3	9.0	14.3	12.0	8.4	10.5	4.5	6.7	5.4
LWP[g] max 10m profile variance	21.1	39.6	47.7	59.0	67.3	45.3	63.6	50.3	26.2	50.7
LWP max 30m profile variance	1087.8	900.2	674.6	955.7	226.8	438.8	200.3	411.2	222.9	220.1
RWP max 3m profile variance	9.9	7.7	8.4	12.0	10.4	5.7	5.5	5.9	12.7	9.6
RWP max10m profile variance	61.0	26.4	32.6	55.0	19.1	24.0	47.0	58.9	35.0	27.1
RWP max 30m profile variance	986.4	1108.8	889.3	1046.1	469.3	141.9	350.6	434.9	161.5	172.3
Max NAASRA	126	119	146	189	143	147	189	165	162	188

Table 3.1 (cont.) Candidate bridge characteristics

Feature	Bridge name									
	Waikanae River		Manawatu (Whirokino)		Rangitikei (Bulls)		Mangaone Stream		Lynch's Stream	
	Inc[d]	Dec[e]	Inc	Dec	Inc	Dec	Inc	Dec	Inc	Dec
Year	1963		1944		1949		1929		1929	
SH[a]	1N		1N		1N		56		57	
RP[b]	1012		954		926		22		36	
Disp[c]	5.16		13.0		1.27		1.21		2.83	
Cross-section of superstructure	Slab		Beam and slab non-composite		Beam and slab composite		Beam and slab non-composite		Beam and slab non-composite	
Long section of superstructure	Continuous spans		Suspended spans		Suspended spans		Continuous spans		Simple spans	
Beam type	?		Plate girders		?		I-beams		I-beams	
Deck material	Reinforced concrete		Reinforced concrete		Reinforced concrete		Reinforced concrete		Reinforced concrete	
Superstructure material[f]	Concrete, precast and pre-tensioned		Steel		Steel		Steel		Steel	
Length	79.9		180.9		420.1		18.2		12.6	
Spans	5		7		16		2		1	
LWP max 3m profile variance	15.2	4.8	6.6	9.6	5.8	2.0	2.9	2.6	2.7	7.2
LWP[g] max 10m profile variance	136.0	55.2	57.5	40.9	32.5	29.3	7.5	33.0	11.1	41.8
LWP max 30m profile variance	1768.3	676.4	826.8	1076.2	384.1	646.1	97.9	432.3	259.0	1091.9
RWP max 3m profile variance	5.3	11.5	6.6	6.8	6.7	3.3	1.8	3.2	2.3	2.6
RWP max 10m profile variance	66.3	45.3	50.4	43.6	52.4	28.1	7.6	28.9	7.4	41.5
RWP max 30m profile variance	1506.2	1004.8	879.9	1100.3	240.6	470.3	177.5	430.7	369.2	1299.6
Max NAASRA	213	154	159	171	177	119	92	114	97	110

Notes to table 3.1:

a SH = State Highway

b RP = route position (km)

c Disp = distance from route position marker

d Inc = increasing direction as defined in RAMM

e Dec = decreasing direction as defined in RAMM

f Different superstructure materials have been highlighted in different typefaces

g Profile variance data has been highlighted in grey fill if a figure is above the UK ride quality thresholds from table 2.1, and bold typeface if below this threshold. Maximum NAASRA data has been highlighted in bold if less than 100, and grey fill if greater than 150. LWP = left wheelpath; RWP = right wheelpath

Having reduced the number of bridges with characteristics considered appropriate to the project steering group to 20, the practicalities and logistics of carrying out full-scale measurements on each of the bridges were considered. Reviews of the aerial photos and video records for each of the 20 bridges, alongside site visits and inspections of most of them, reduced the final number to seven. These are listed in table 3.2. Photos of each of the bridges are given in figures 3.1 to 3.7, and plots of the profile variance data for the bridges and approaches are given in figures 3.8 to 3.21.

Table 3.2 Bridge characteristics – final selection for full-scale measurements

Bridge name*		Year	Beam type	Deck material	Super-structure material	Length (m)	Spans	Profile variance (max)**						Max NAASRA
								LWP			RWP			
								3m	10m	30m	3m	10m	30m	
Tauherenikau	↑	1971	Double hollow core units	Reinforced concrete	Concrete, cast in situ and reinforced	126.5	10	1.9	10.8	152.3	2.2	11.4	162.9	74
	↓							1.8	6.8	79.5	3.1	7.2	57.3	94
Waiohine	↑	2006	Double hollow core units	Pre-stressed concrete	Concrete, precast and pre-tensioned	93.85	5	3.3	50.4	443.1	4.7	32.6	597.8	116
	↓							3.5	13.2	338.0	10.2	59.9	1984.1	138
Waingawa	↑	1991	U-beams	Reinforced concrete	Concrete, precast and pre-tensioned	136.6		7.0	42.3	151.5	9.8	28.8	430.2	174
	↓							10.5	35.3	142.2	11.3	37.8	96.7	198
Mangatewai-iti Stream	↑	1983	I-beams	Reinforced concrete	Concrete, precast and pre-tensioned	85.2	4	4.6	21.1	1087.8	9.9	61.0	986.4	126
	↓							6.3	39.6	900.2	7.7	26.4	1108.8	119
Mangatewai-nui River	↑	1988	Other	Reinforced concrete	Steel	102.3	3	9.0	47.7	674.6	8.4	32.6	889.3	146
	↓							14.3	59.0	955.7	12.0	55.0	1046.1	189
Waikanae River	↑	1963	?	Reinforced concrete	Concrete, precast and pre-tensioned	79.9	5	15.2	136.0	1768.3	5.3	66.3	1506.2	213
	↓							4.8	55.2	676.4	11.5	45.3	1004.8	154
Manawatu (Whirokino)	↑	1944						6.6	57.5	826.8	6.6	50.04	879.9	159
	↓							9.6	40.9	1076.2	6.8	43.6	1100.3	171

*↑ indicates increasing direction; ↓ indicates decreasing direction

** Profile variance data has been highlighted in bold typeface if a figure is above the UK ride quality thresholds from table 2.1, and grey fill if below this threshold. Maximum NAASRA data has been highlighted in bold if less than 100, and grey fill if greater than 150. LWP = left wheelpath; RWP = right wheelpath.

3.3 Selected bridges

3.3.1 Photographs

Figure 3.1 Tauherenikau bridge looking north (decreasing direction)

Figure 3.2 Waiohine bridge looking south (increasing direction)

Figure 3.3 Waingawa bridge looking north (decreasing direction)

Figure 3.4 Mangatewai-iti bridge looking north (decreasing direction)

Figure 3.5 Mangatewai-nui bridge looking north (decreasing direction)

Figure 3.6 Waikanae bridge looking south (increasing direction)

Figure 3.7 Manawatu (Whirokino) bridge looking south (increasing direction)

3.3.2 Profile variance data plots

The profile variance data has been plotted on common vertical (y) axes so that the levels can be readily compared between bridges. In each figure, solid lines indicate the left wheelpath, while dashed lines indicate the right wheelpath.

Figure 3.8 Roughness and profile variance of Tauherenikau bridge (increasing direction: southbound)

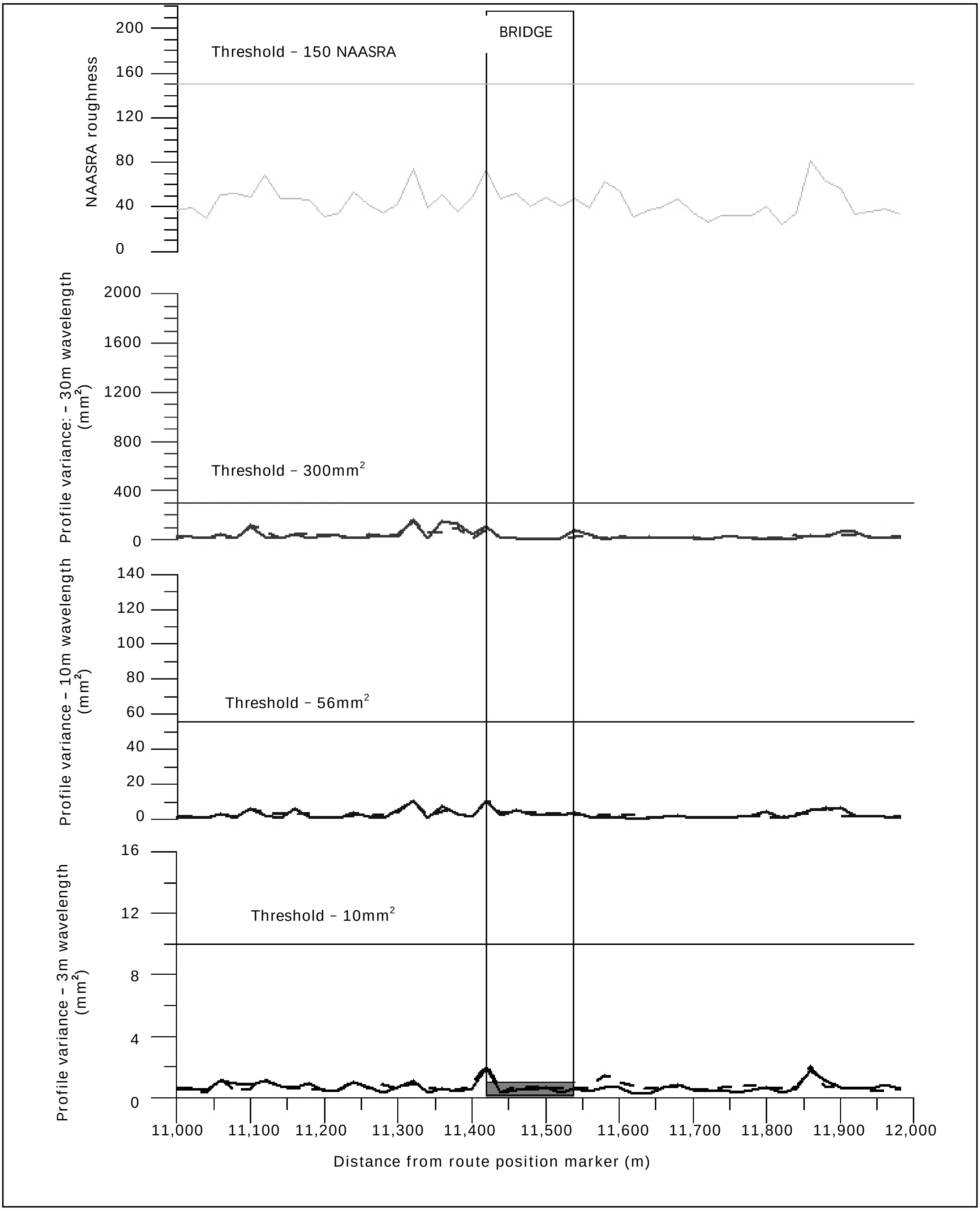


Figure 3.9 Roughness and profile variance of Tauherenikau bridge (decreasing direction: northbound)

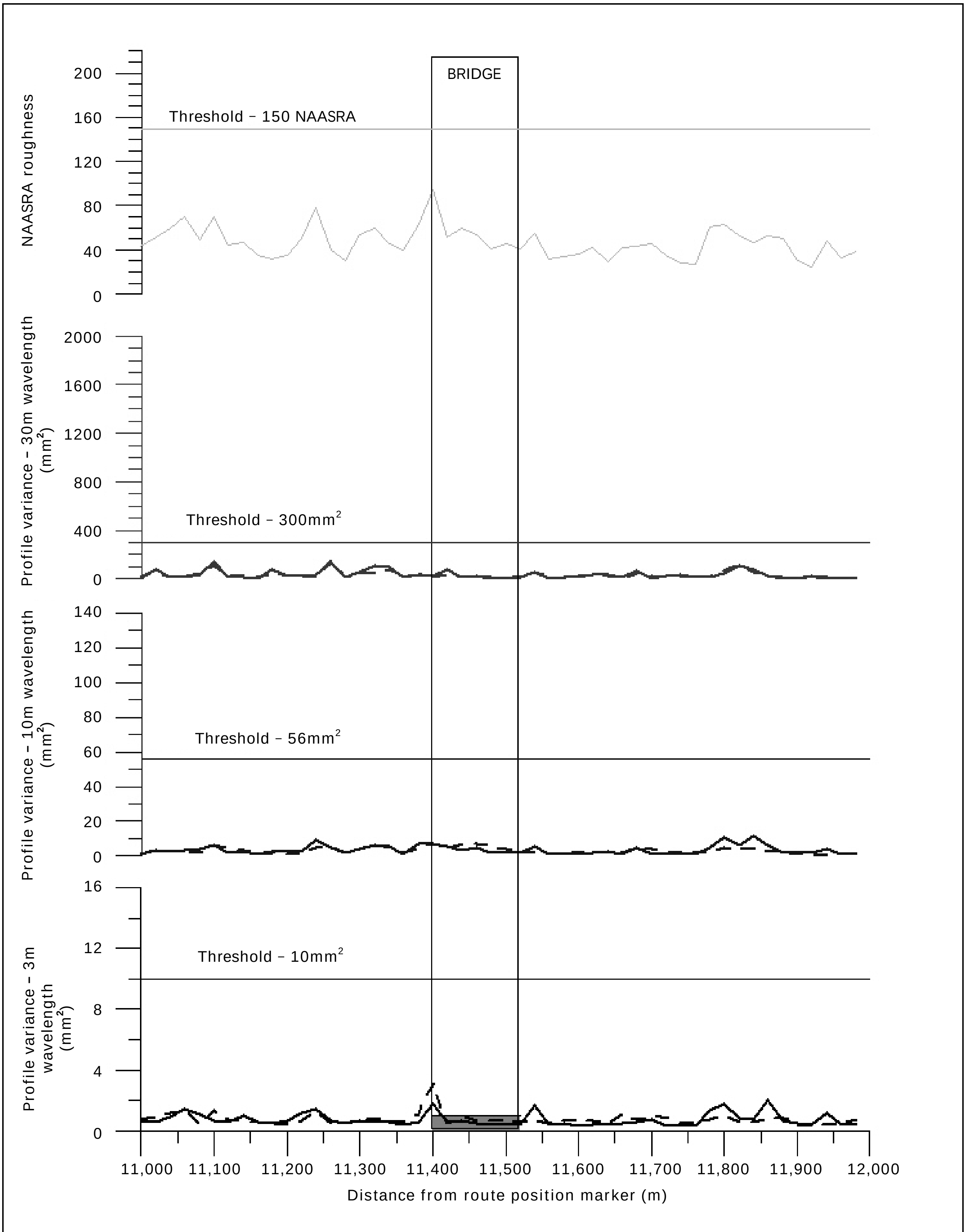


Figure 3.10 Roughness and profile variance of Waiohine bridge (increasing direction: southbound)

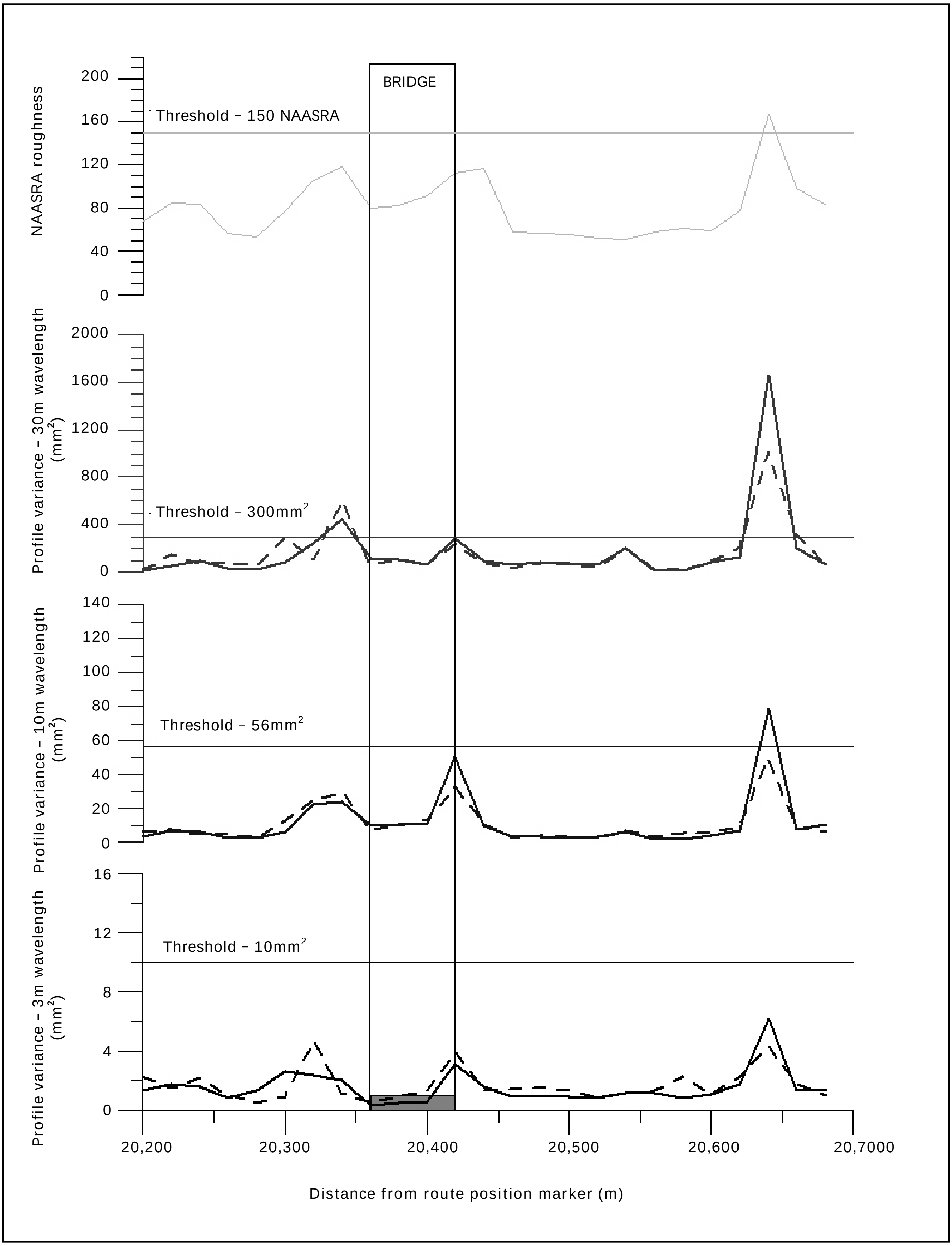


Figure 3.11 Roughness and profile variance of Waiohine bridge (decreasing direction: northbound)

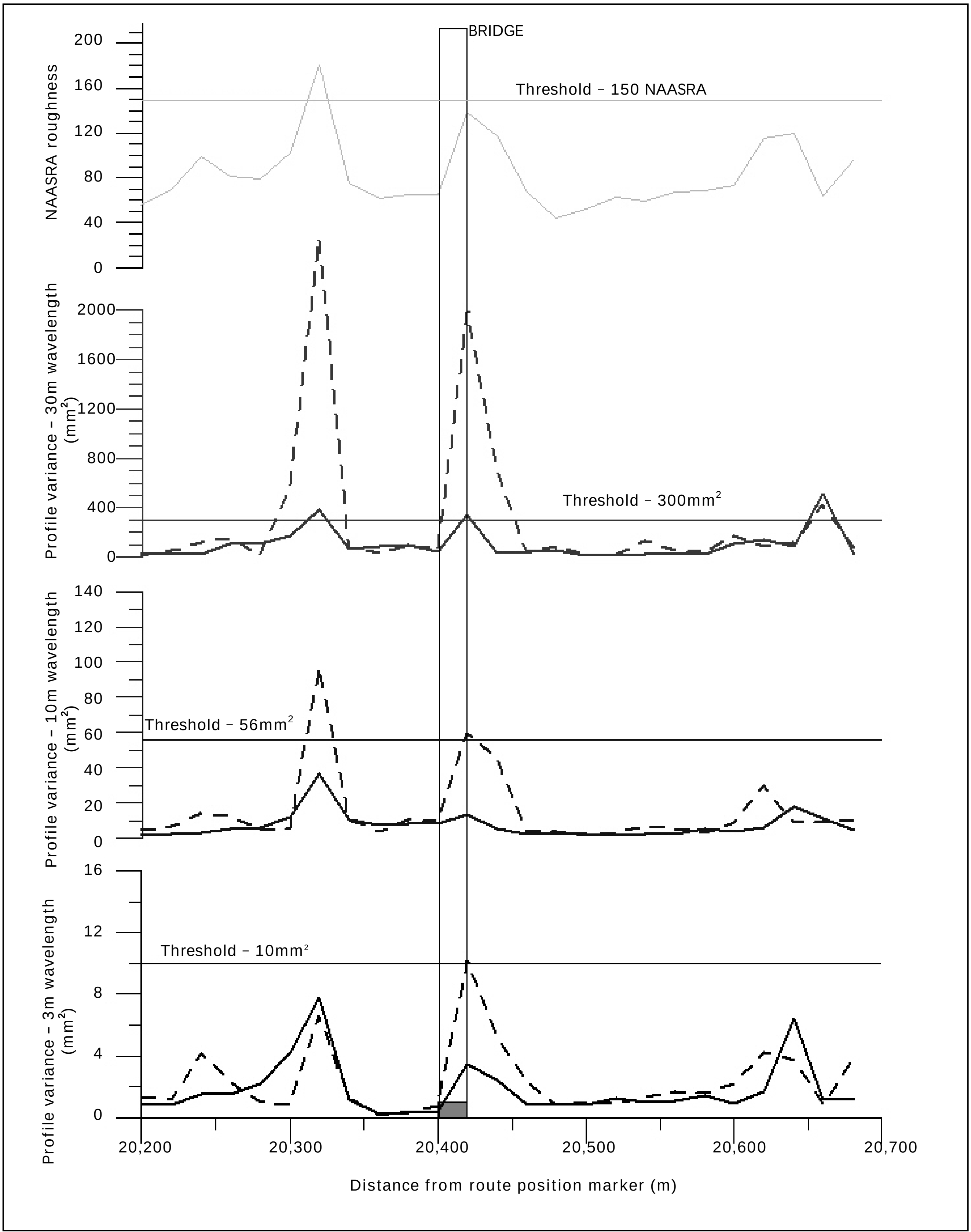


Figure 3.12 Roughness and profile variance of Waingawa bridge (increasing direction: southbound)

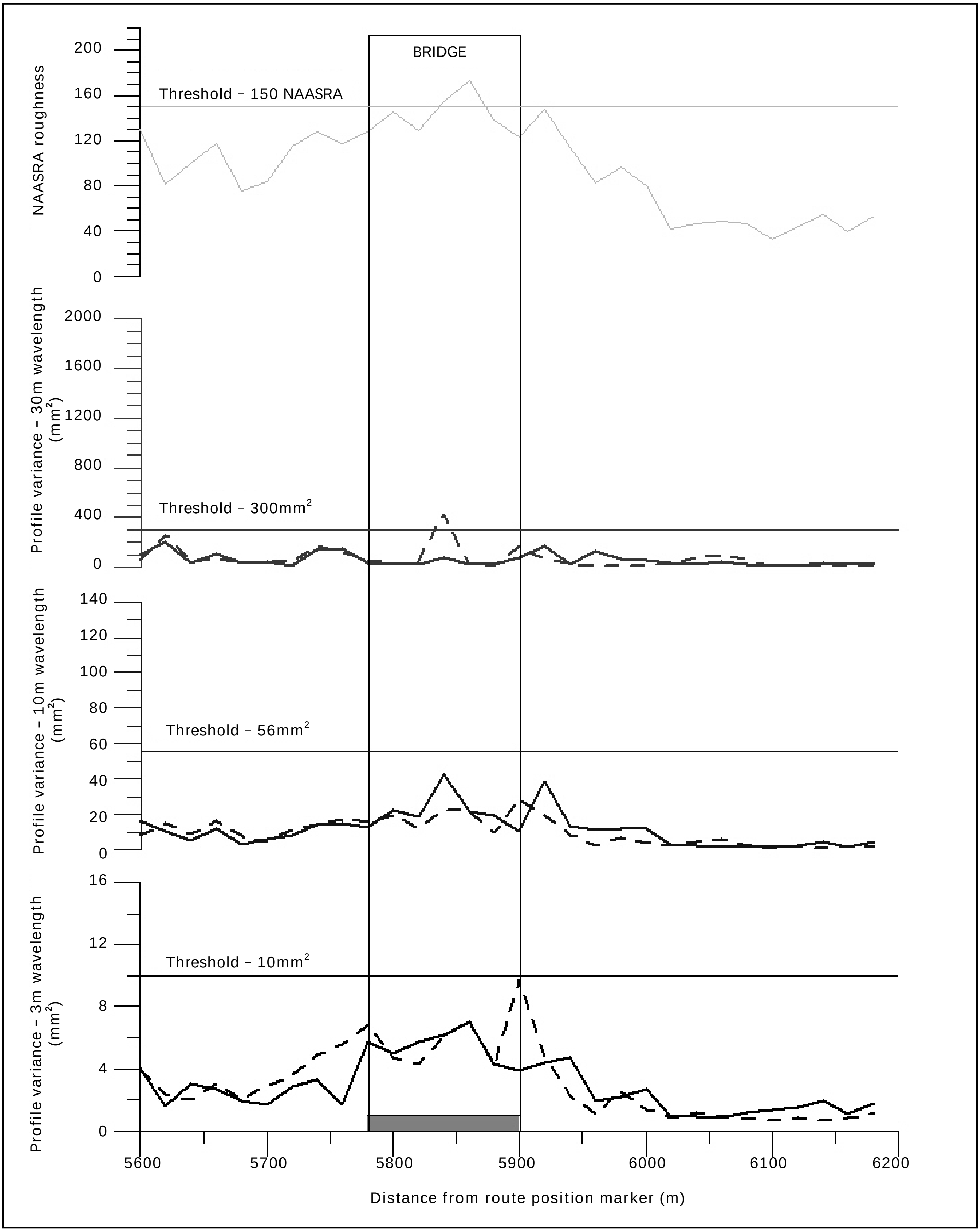


Figure 3.13 Roughness and profile variance of Waingawa bridge (decreasing direction: northbound)

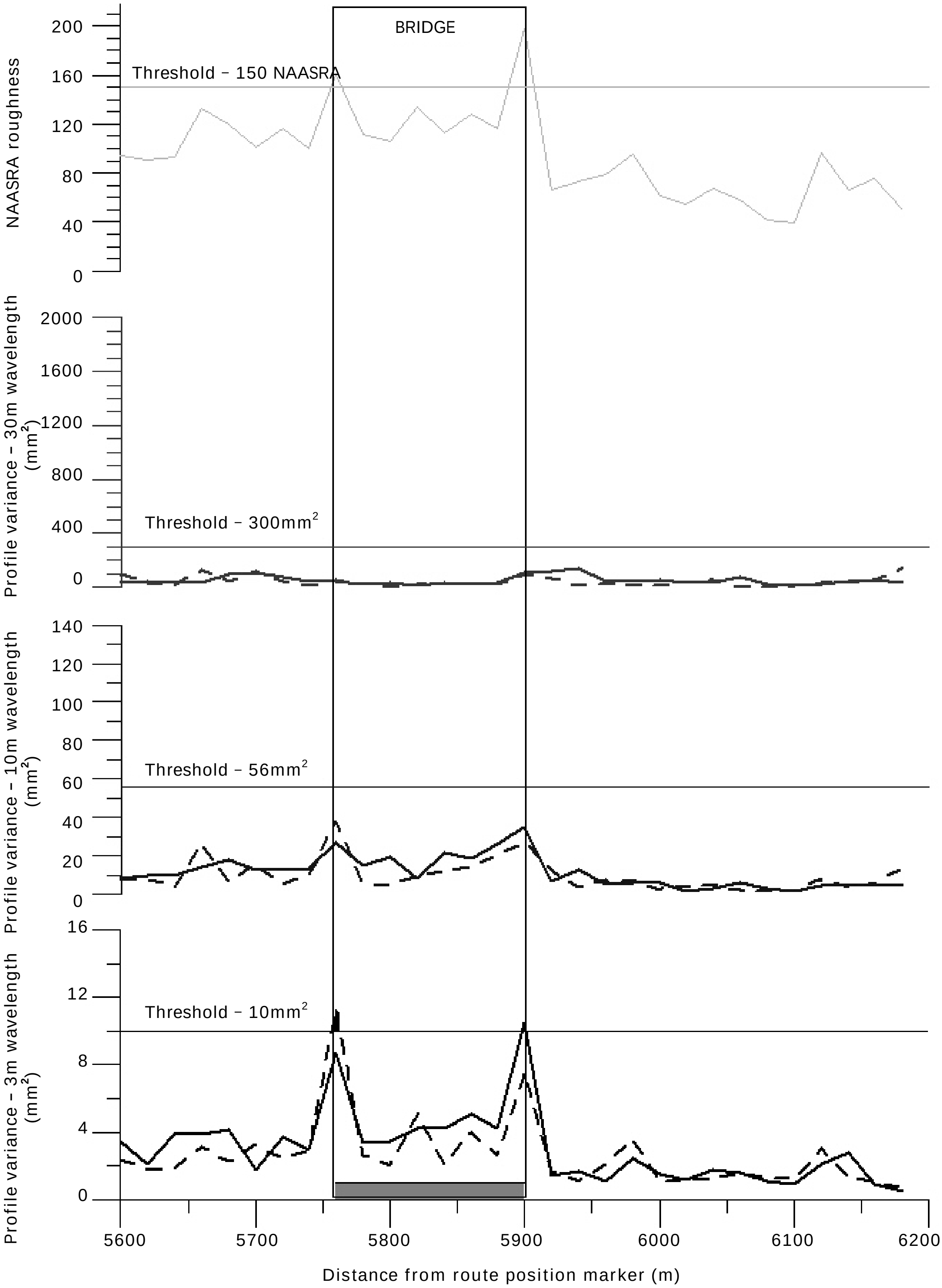


Figure 3.14 Roughness and profile variance of Mangatewai-iti bridge (increasing direction: southbound)

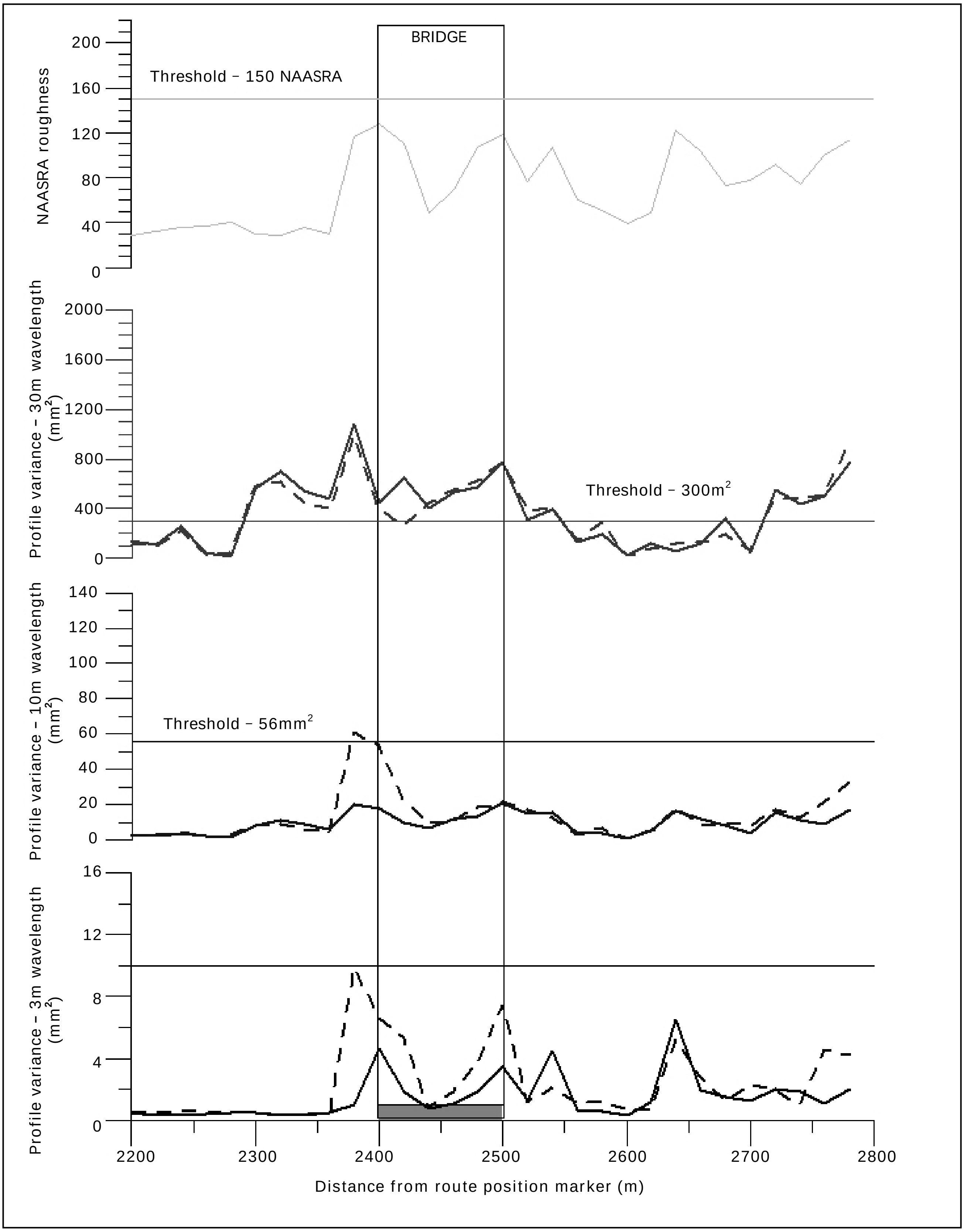


Figure 3.15 Roughness and profile variance of Mangatewai-iti bridge (decreasing direction: northbound)

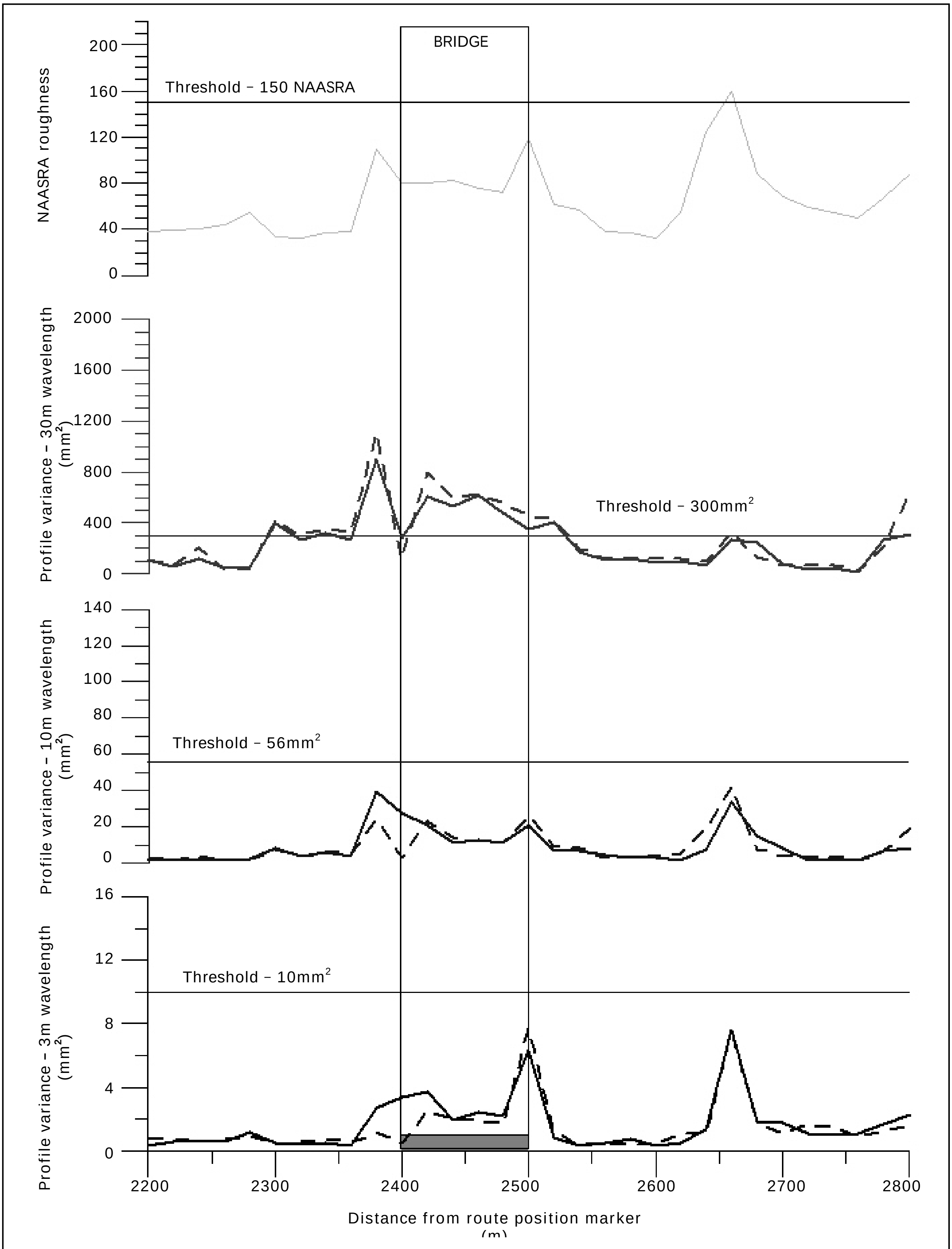


Figure 3.16 Roughness and profile variance of Mangatewai-nui bridge (increasing direction: southbound)

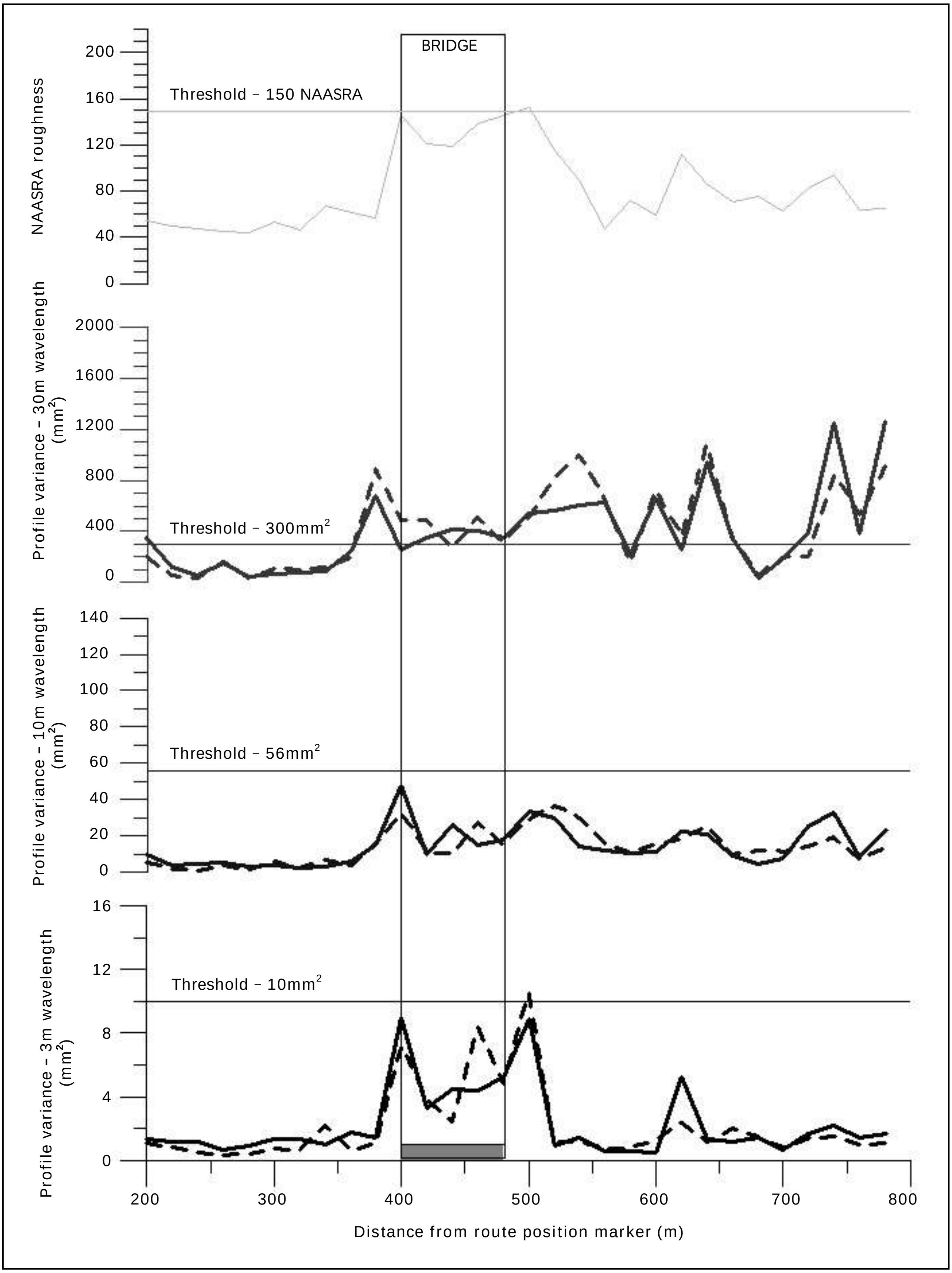


Figure 3.17 Roughness and profile variance of Mangatewai-nui bridge (decreasing direction: northbound)

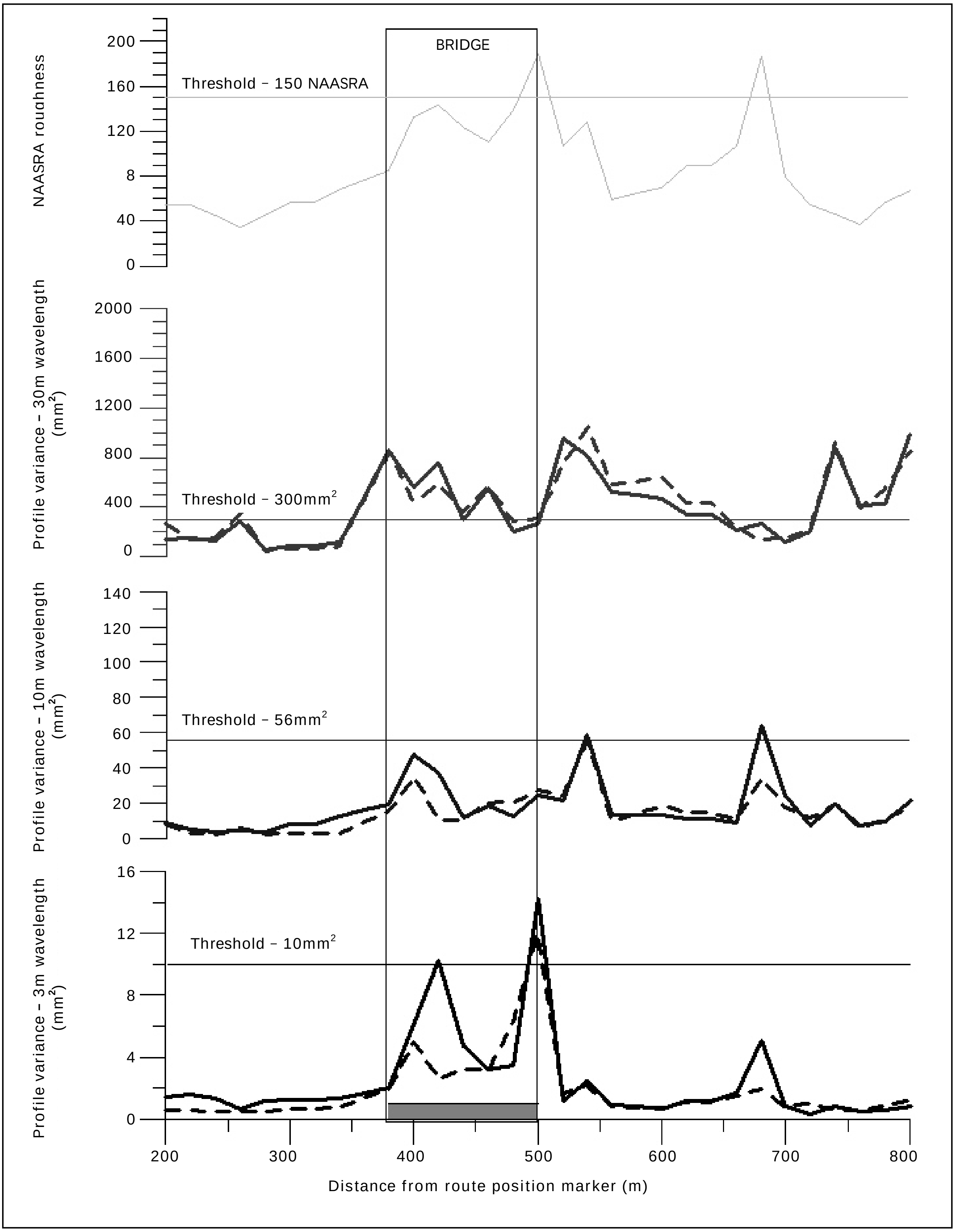


Figure 3.18 Roughness and profile variance of Waikanae bridge (increasing direction: southbound)

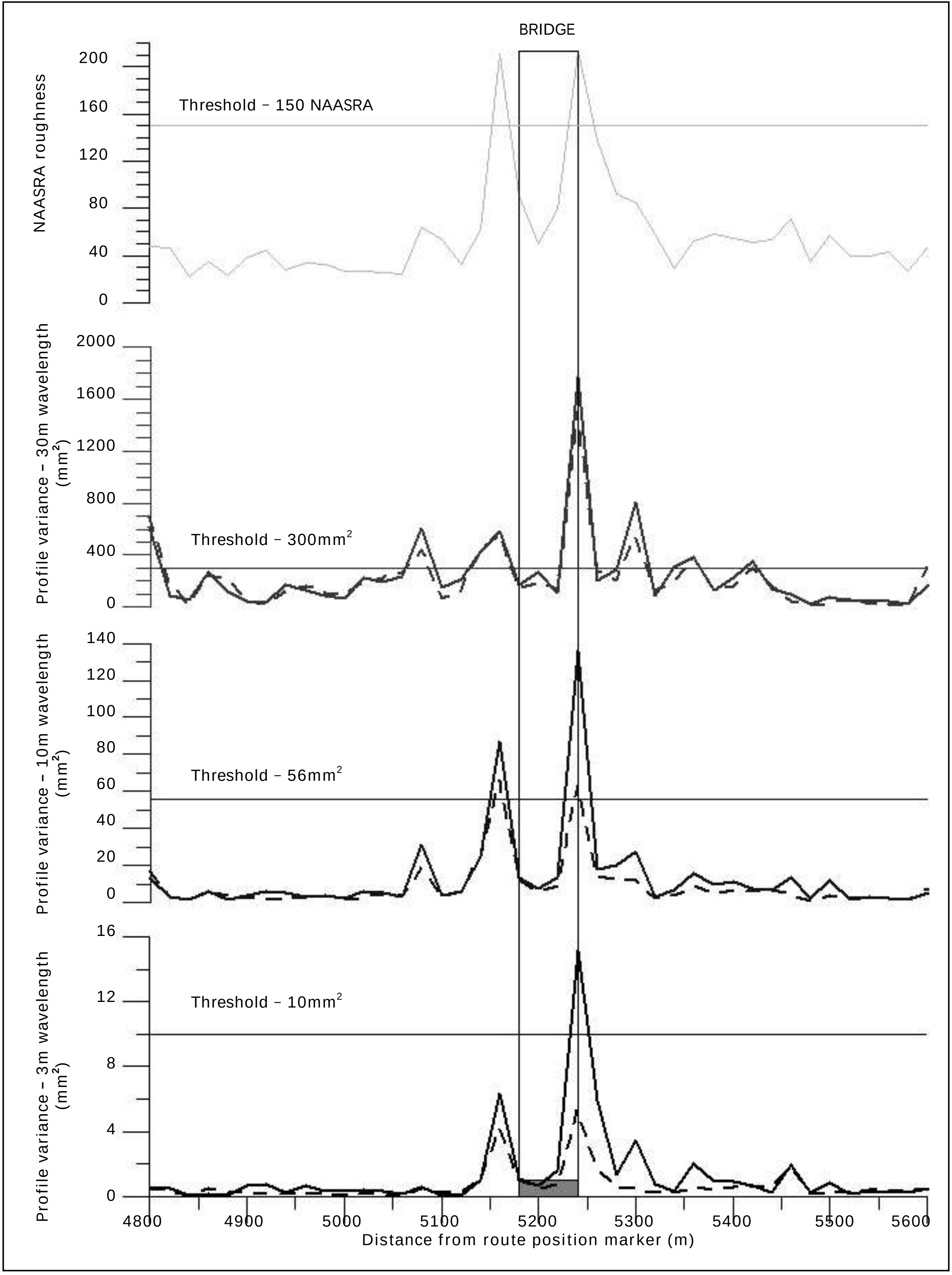


Figure 3.19 Roughness and profile variance of Waikanae bridge (decreasing direction: northbound)

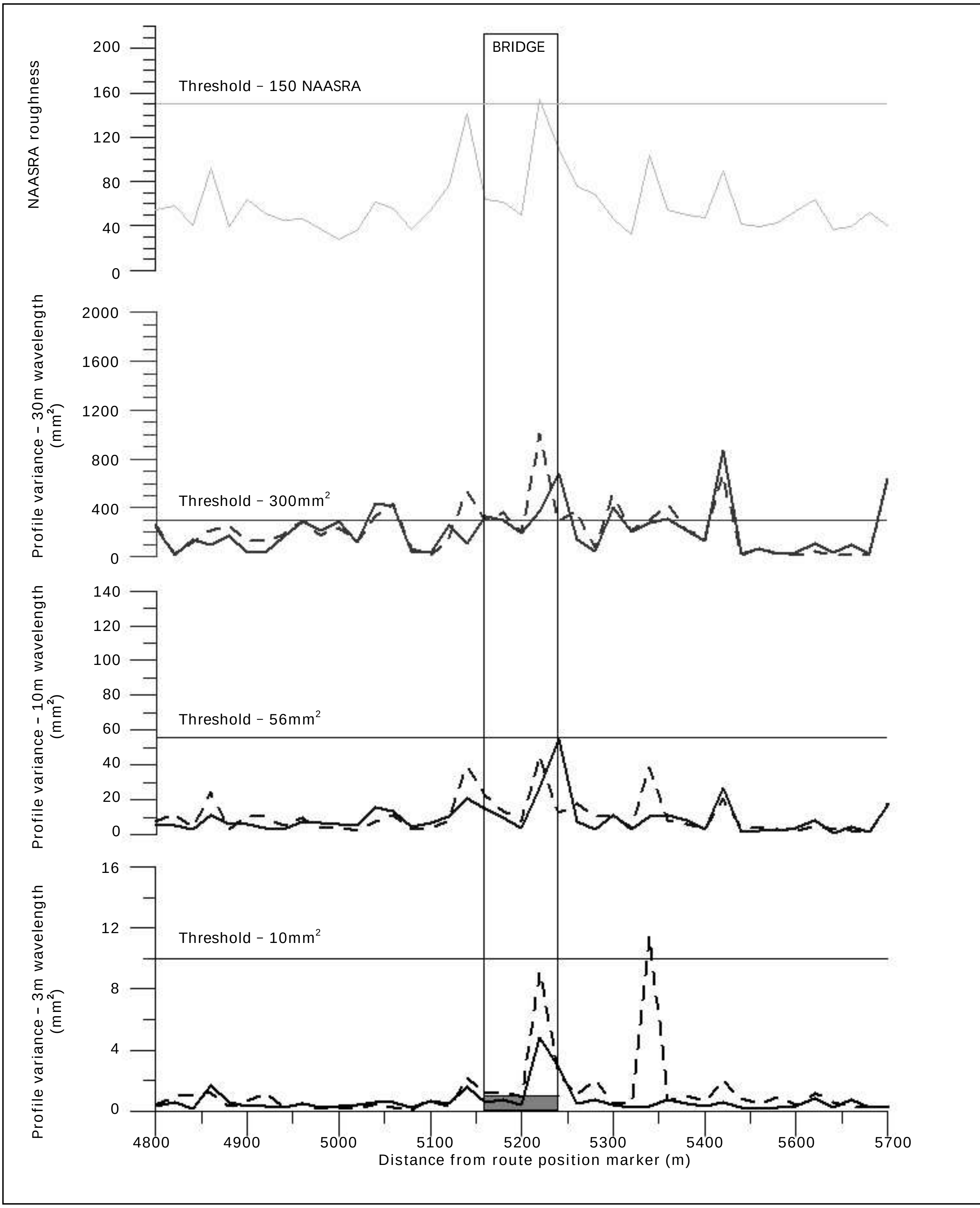


Figure 3.20 Roughness and profile variance of Manawatu (Whirokino) bridge (increasing direction: southbound)

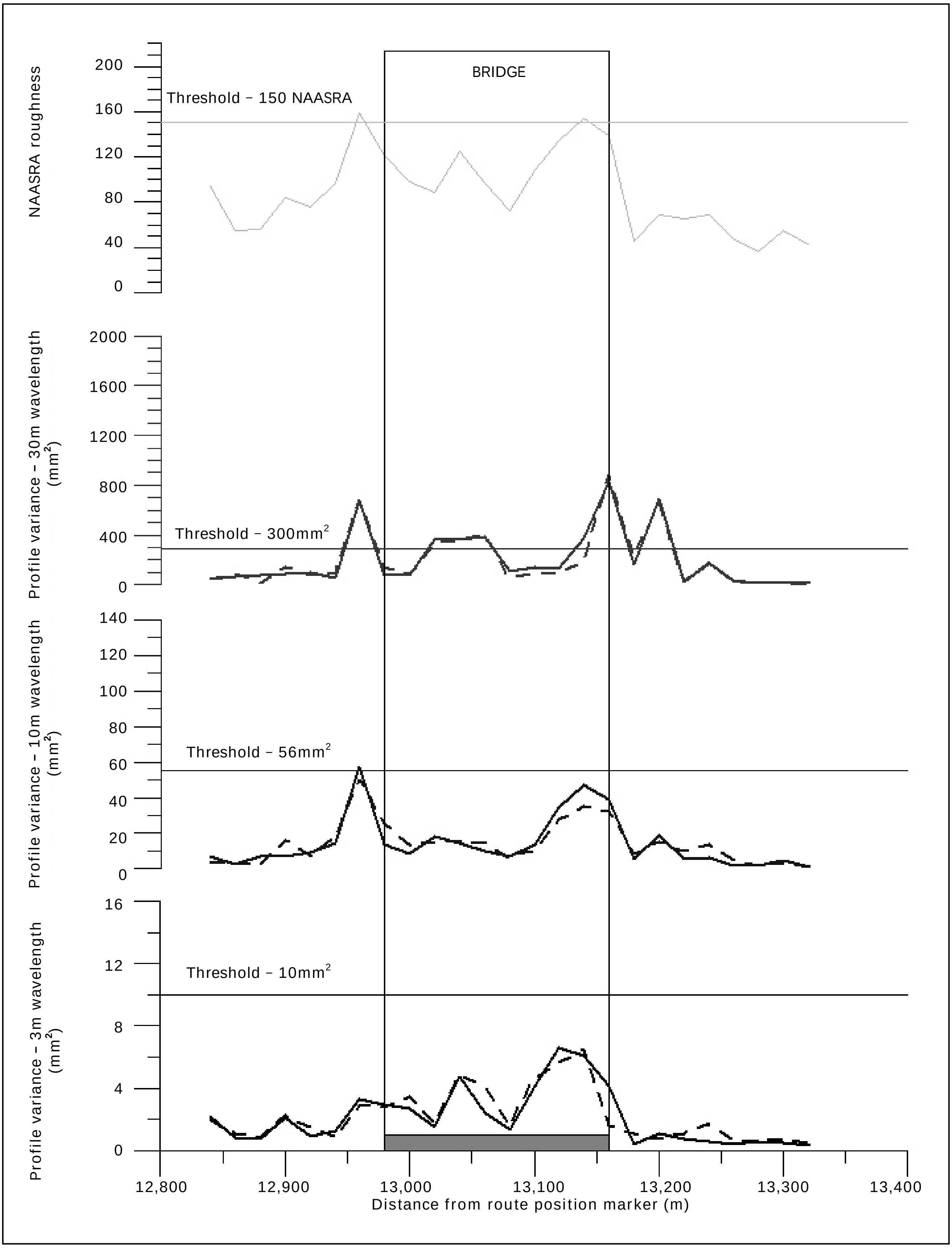
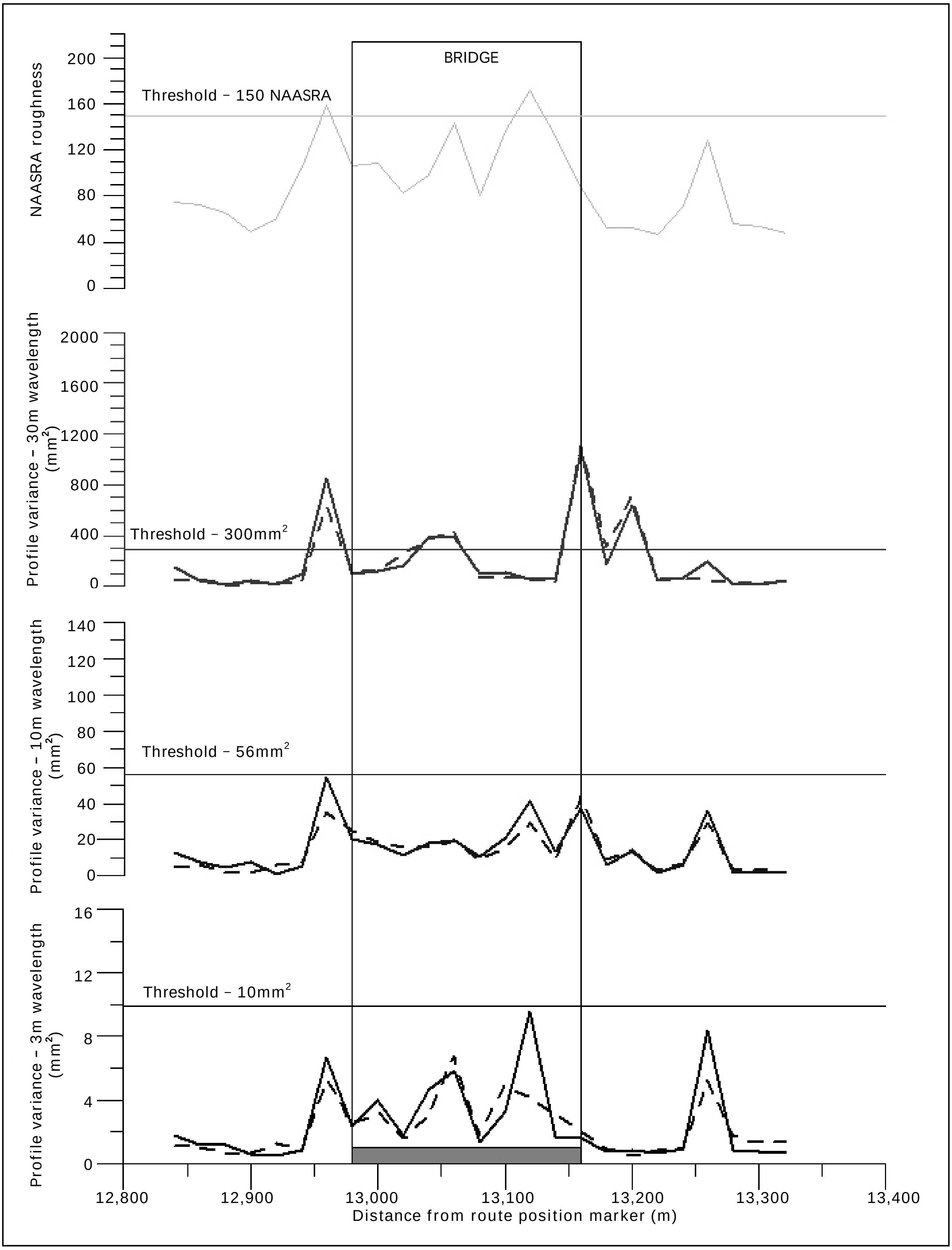


Figure 3.21 Roughness and profile variance of Manawatu (Whirokino) bridge (decreasing direction: northbound)



The following observations can be made from table 3.2 and figures 3.1 to 3.21:

- The bridges chosen for monitoring have a wide range of profile variance across the three different wavelengths, and also different profile variance levels at both ends of the bridge and in the different wheelpaths. This is important because it means that differences between the responses of any individual bridge in different directions can be ascribed to profile or geometry differences, assuming that traffic flows are similar. Similarly, considering a number of bridges with different combinations of profile variance means that correlations can be done to assess the relationships between the dynamic displacements and the different wavelength profile variance values.
- Profile variance levels range from well below the UK Roads Board (2003) thresholds for poor ride quality to well above them.
- The profile variance levels are typically higher on the bridges (including abutments) than on the adjacent road sections.
- Looking at the profile variance peaks, the inaccuracies of the bridge start and end points as identified in the RAMM database appear to be greater in some cases than would be expected, given the 20m distance increments used.

4. Bridge driveover tests

4.1 Background, setup and testing

Given the recommendation of the steering group to monitor the first bridge span accelerations under freestream traffic conditions, it was decided to also carry out simple comparative driveover tests to monitor the response of a vehicle to the roughness and profile variance of the bridges and their approaches. The long wheelbase van that was used for the bridge response accelerometer tests was chosen as providing a level of response somewhat similar to that of a truck.

This van was instrumented with a Crossbow™ six-axis inertial measurement unit comprising three orthogonal electronic gyroscopes to measure pitch, roll and yaw; and three accelerometers to measure longitudinal, lateral and vertical accelerations. The unit was rigidly mounted to the floor of the vehicle. Data from the gyroscopes and accelerometers was recorded at a sample rate of 100Hz using a PC-based data acquisition system.

Data was recorded for driveovers at 100km/h and 70km/h on the Waingawa River bridge, at 70km/h for the Waikanae River bridge and at 100km/h on each of the other five bridges in each direction. Several repeat runs were carried out for each bridge.

It was observed on arrival at the Mangatewai-iti Stream bridge that work appeared to have recently been carried out on both of the bridge's approaches. Subsequent enquiries found that this was the case, with smoothing work being carried out one to two weeks prior to the testing. Accordingly, the actual profile variance levels are expected to be lower than those listed in tables 3.1 and 3.2. These actual levels will not be known until the next annual state highway network survey.

4.2 Data analysis and results

Each of the driveover records was processed to produce time histories of the vertical accelerations for each run. Maximum positive, maximum negative and peak-to-peak accelerations were calculated for each bridge 'drive-on' (immediate approach and abutment). Plots were prepared for one of the test runs over the bridges in each direction. These are presented in figures 4.1 to 4.14. The identified maximum, minimum and root mean squared values for these tests runs are also listed in table 4.1 and compared graphically in figure 4.15. Note that these values are for the vehicle driving onto the bridge, not off it.

Figure 4.1 Driveover response (100km/h) at Tauherenikau bridge (northbound – decreasing direction)

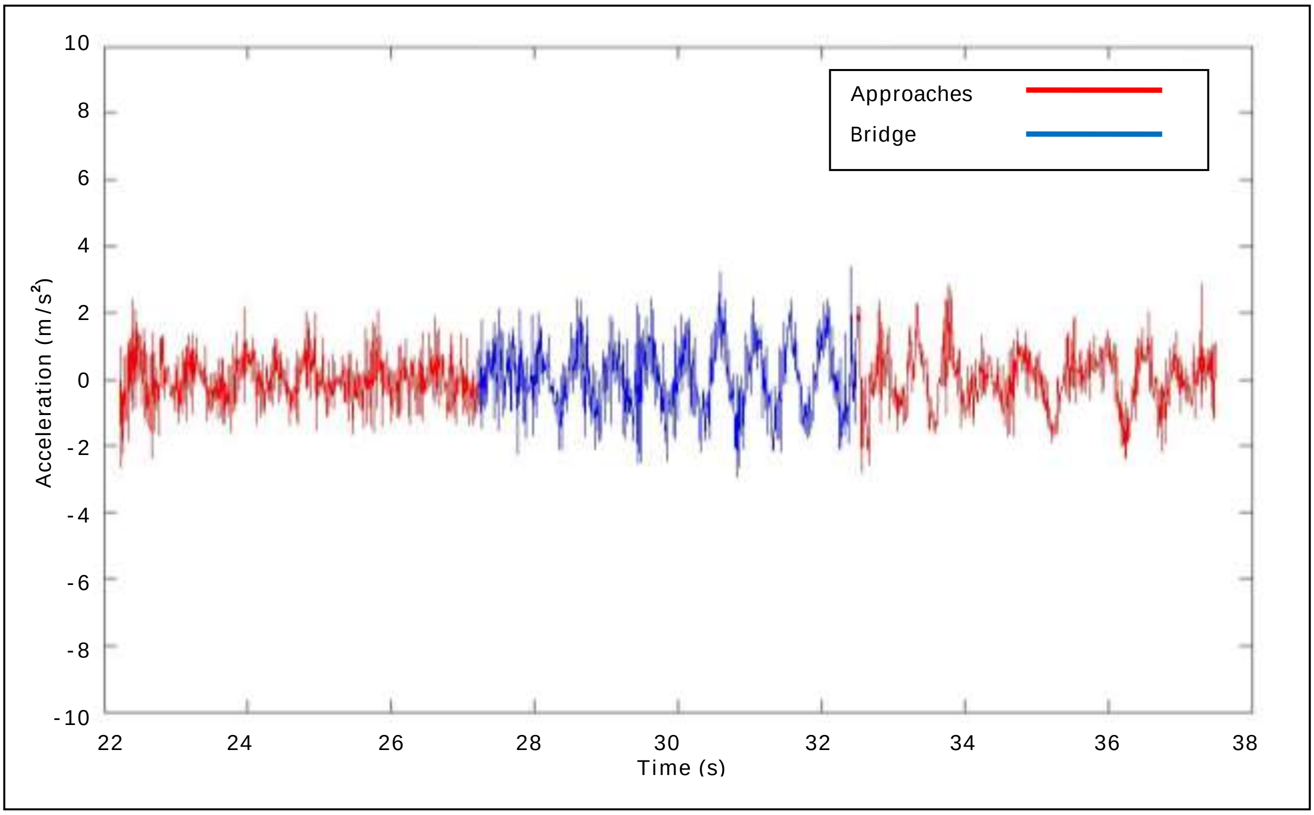


Figure 4.2 Driveover response (100km/h) at Tauherenikau bridge (southbound – increasing direction)

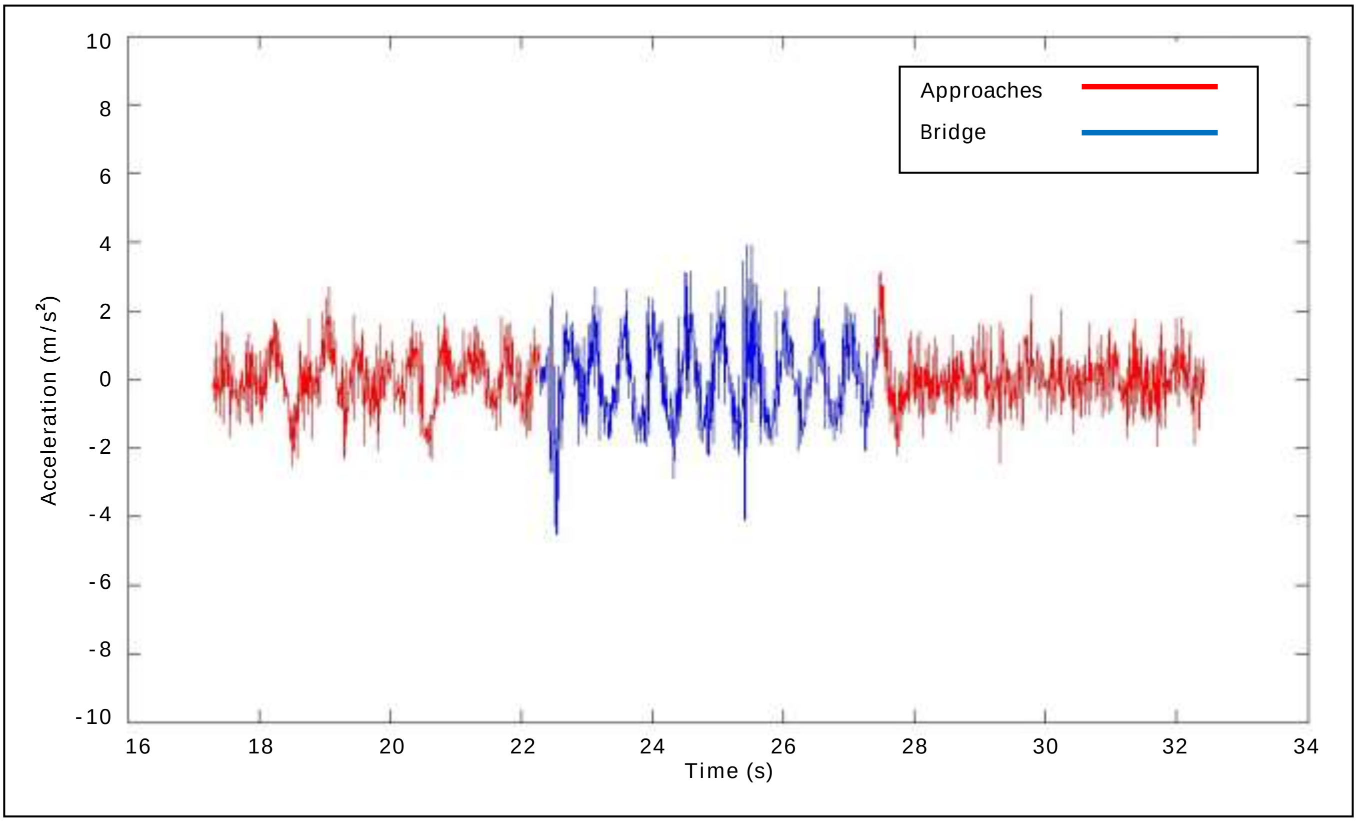


Figure 4.3 Driveover response (100km/h) at Waiohine bridge ((northbound – decreasing direction)

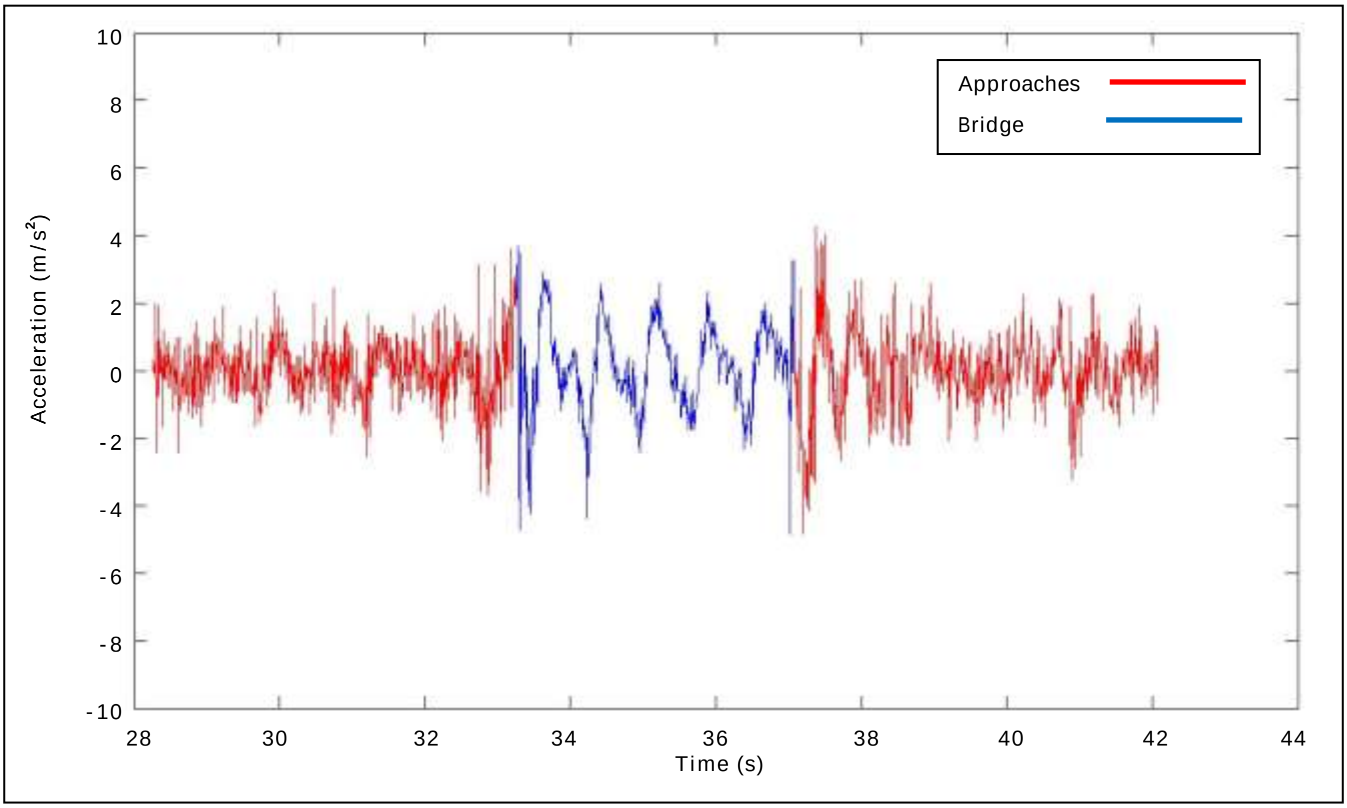


Figure 4.4 Driveover response (100km/h) at Waiohine bridge (southbound – increasing direction)

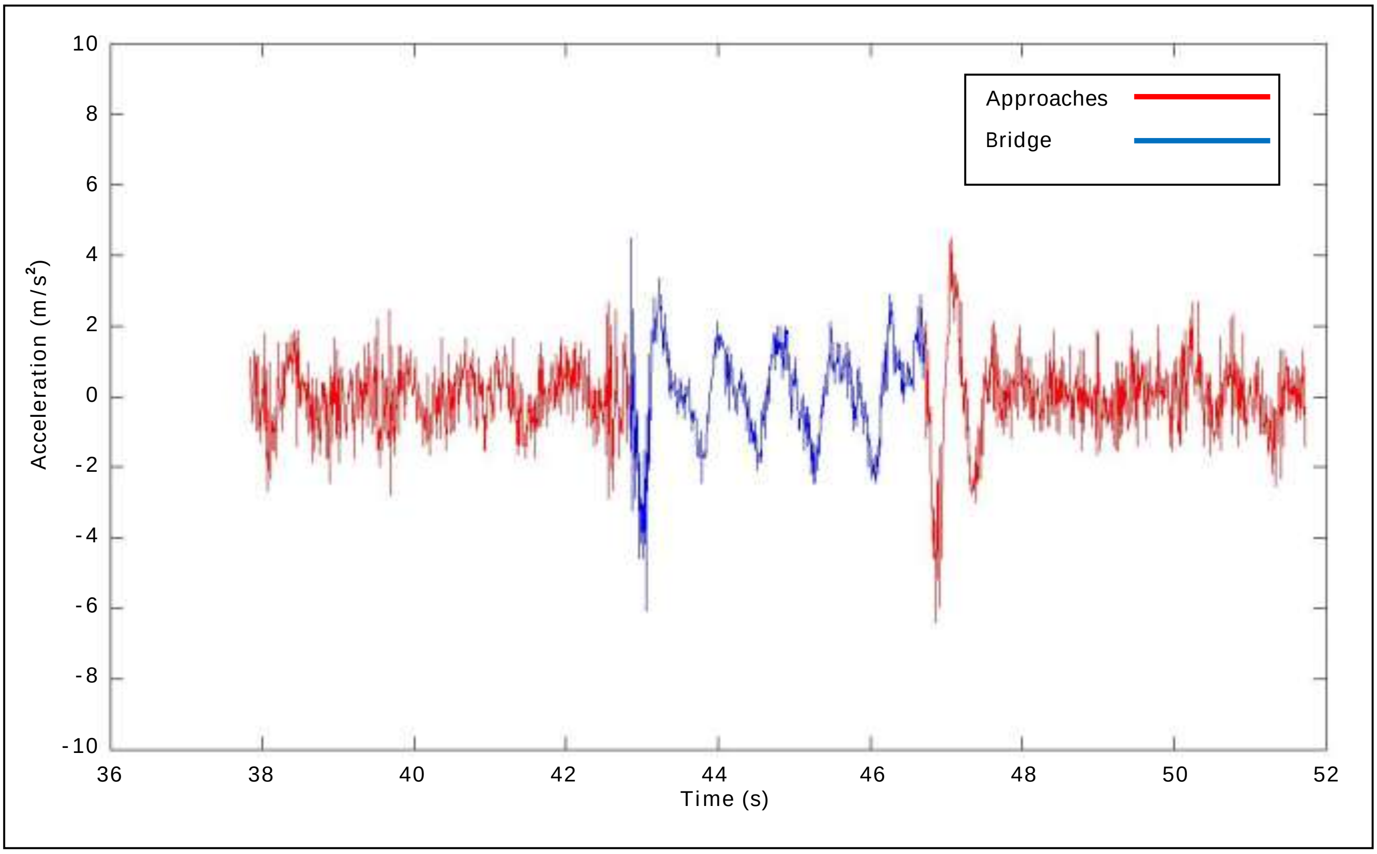


Figure 4.5 Driveover response (100km/h) at Waingawa bridge (northbound – decreasing direction)

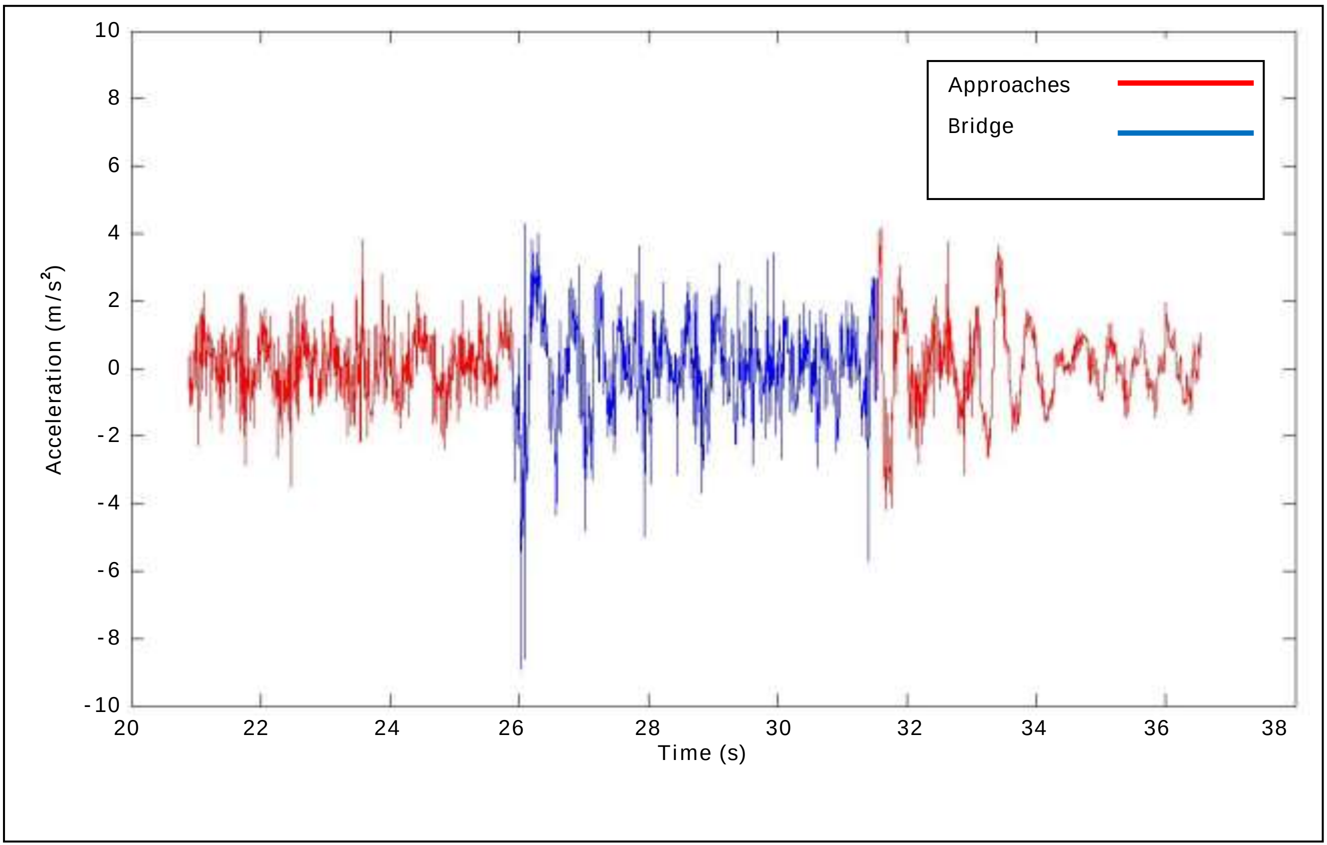


Figure 4.6 Driveover response (70km/h) at Waingawa bridge (southbound – increasing direction)

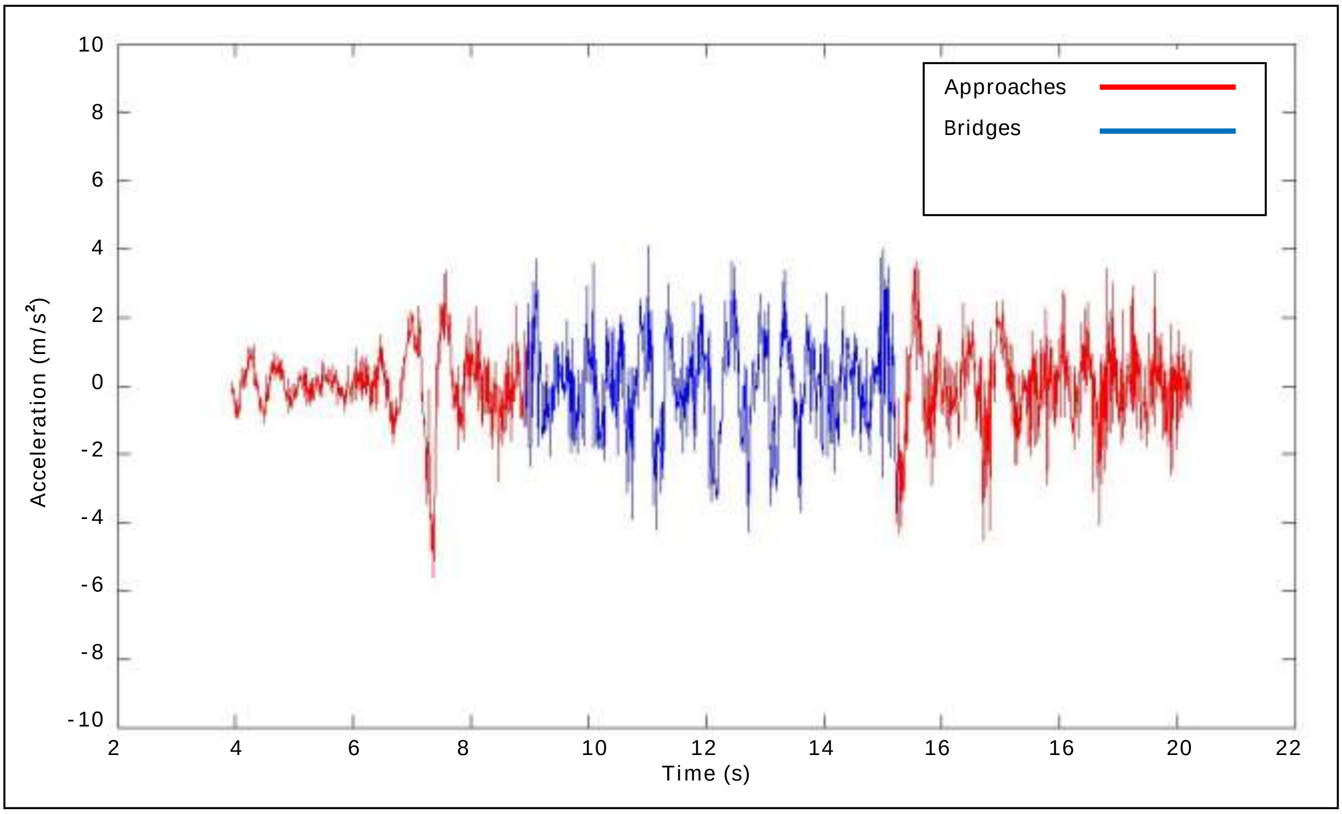


Figure 4.7 Driveover response (100km/h) at Mangatewai-iti bridge (northbound – decreasing direction)

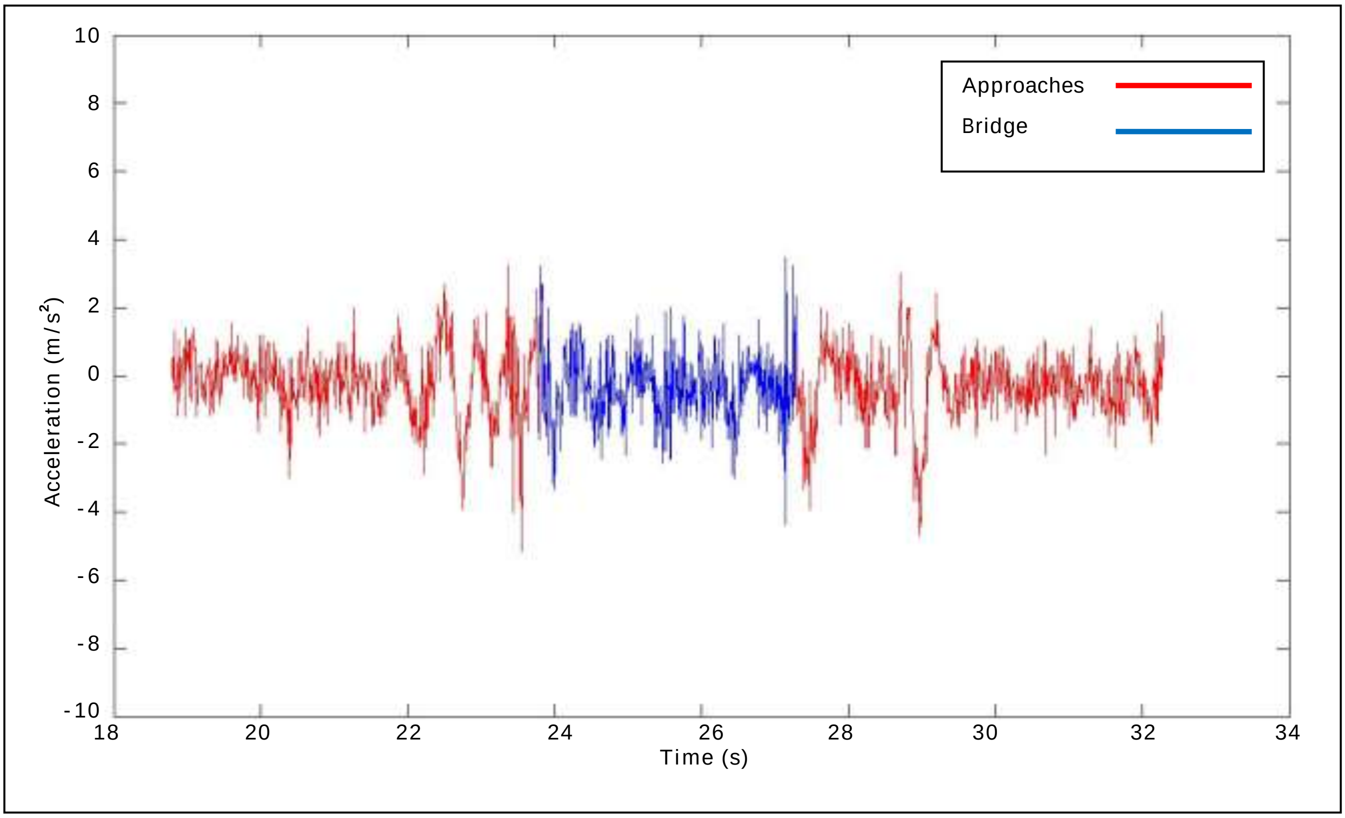


Figure 4.8 Driveover response (100km/h) at Mangatewai-iti bridge (southbound – increasing direction)

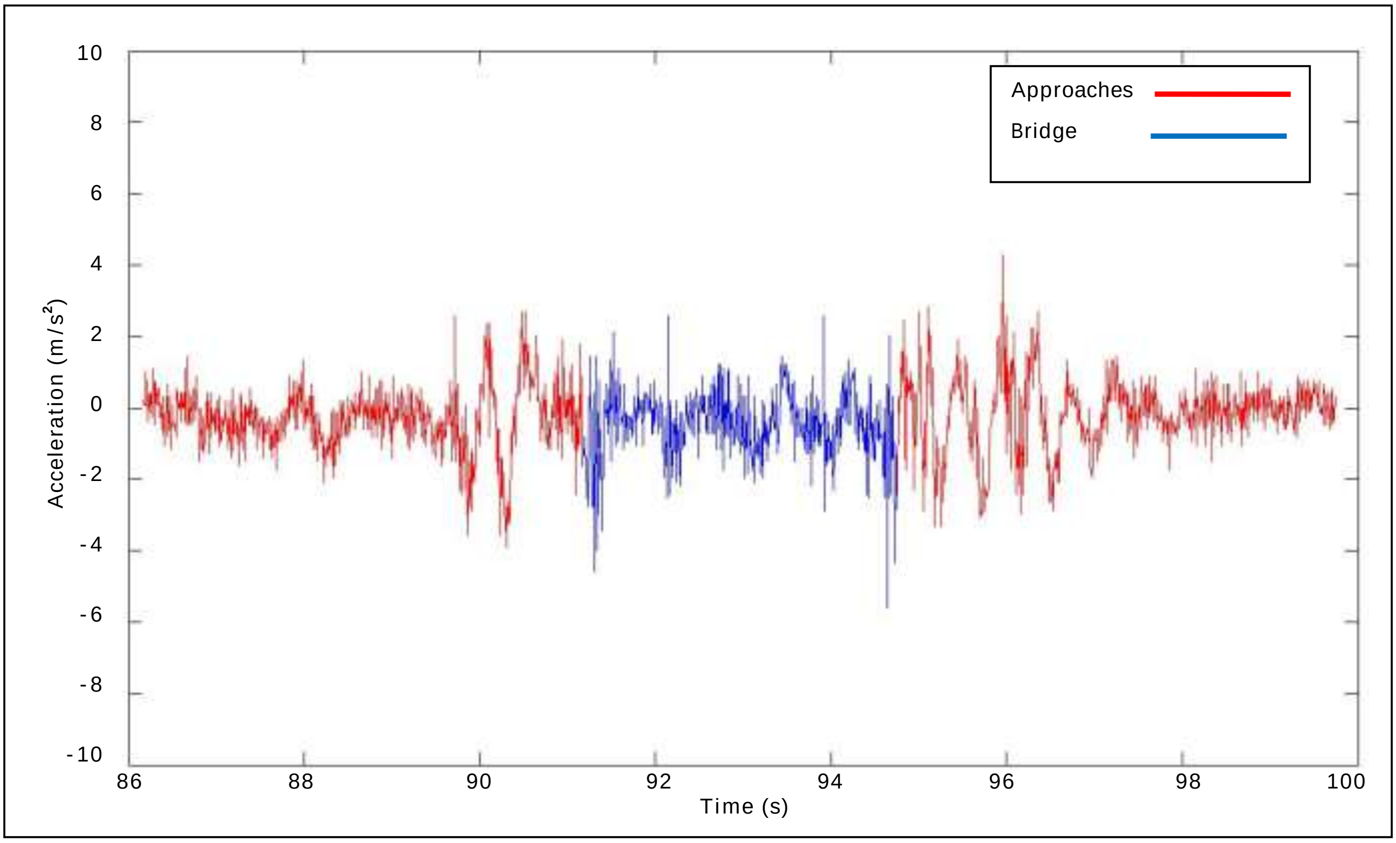


Figure 4.9 Driveover response (100km/h) at Mangatewai-nui bridge (northbound – decreasing direction)

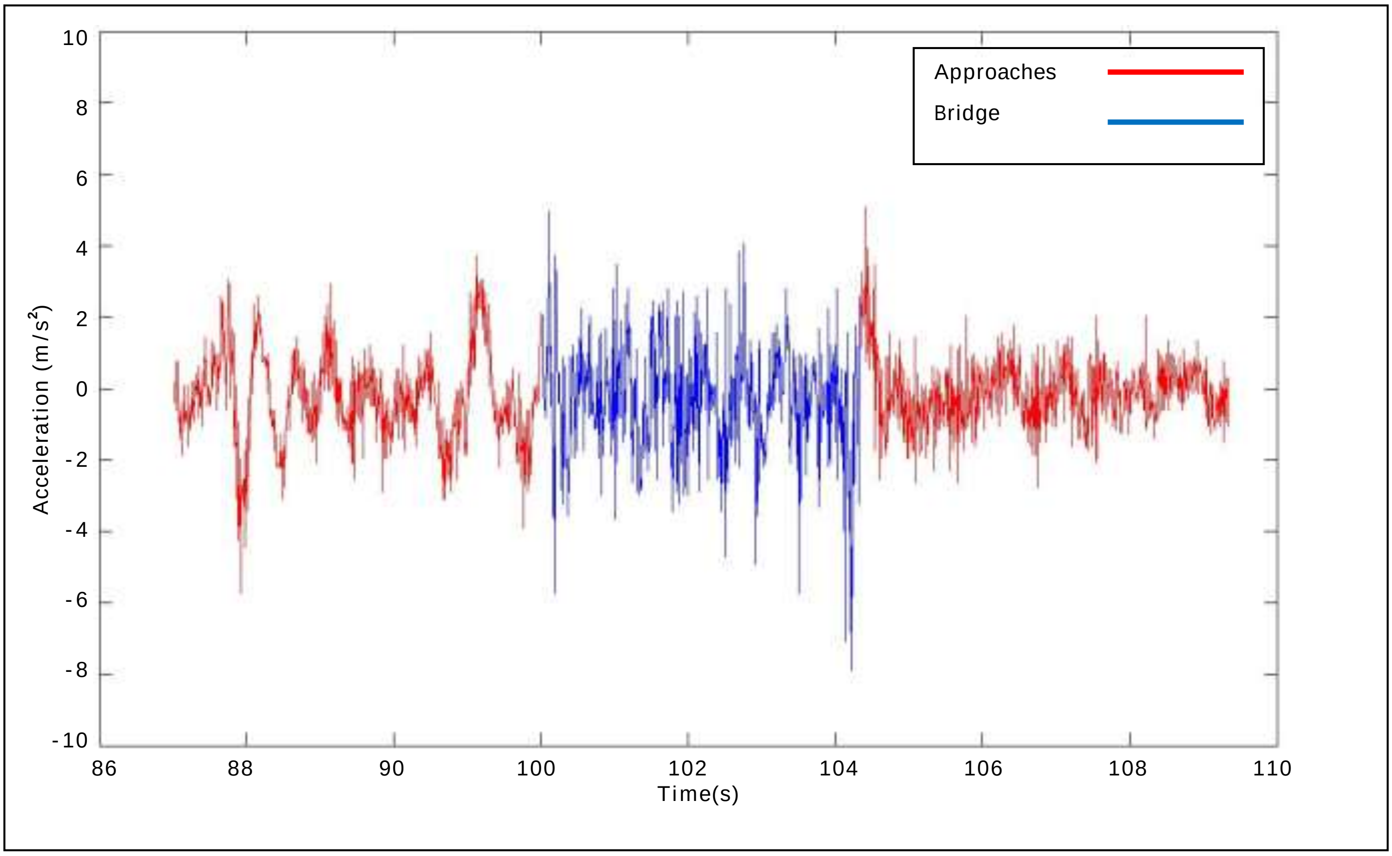


Figure 4.10 Driveover response (100km/h) at Mangatewai-nui bridge (southbound – increasing direction)

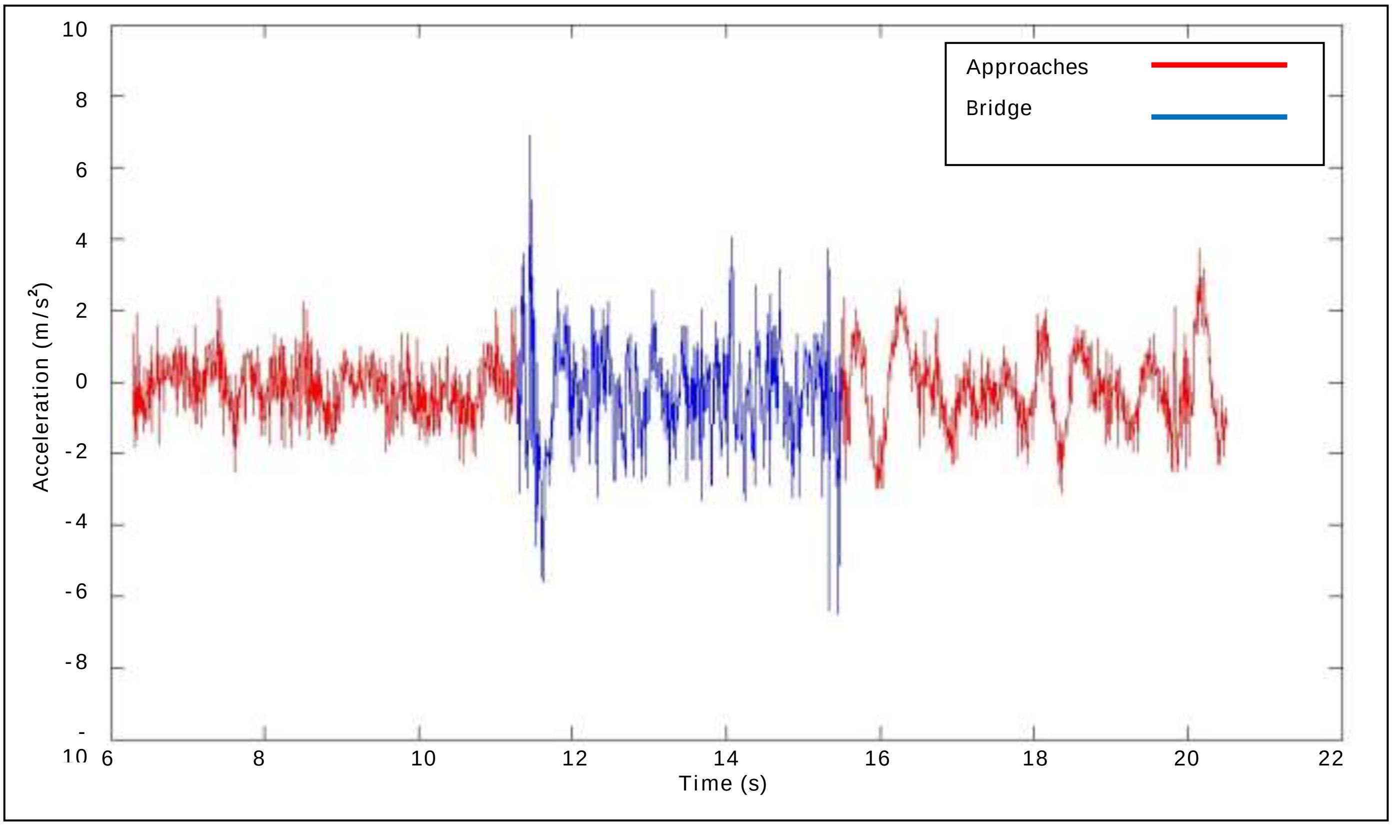


Figure 4.11 Driveover response (70km/h) at Waikanae bridge (northbound – decreasing direction)

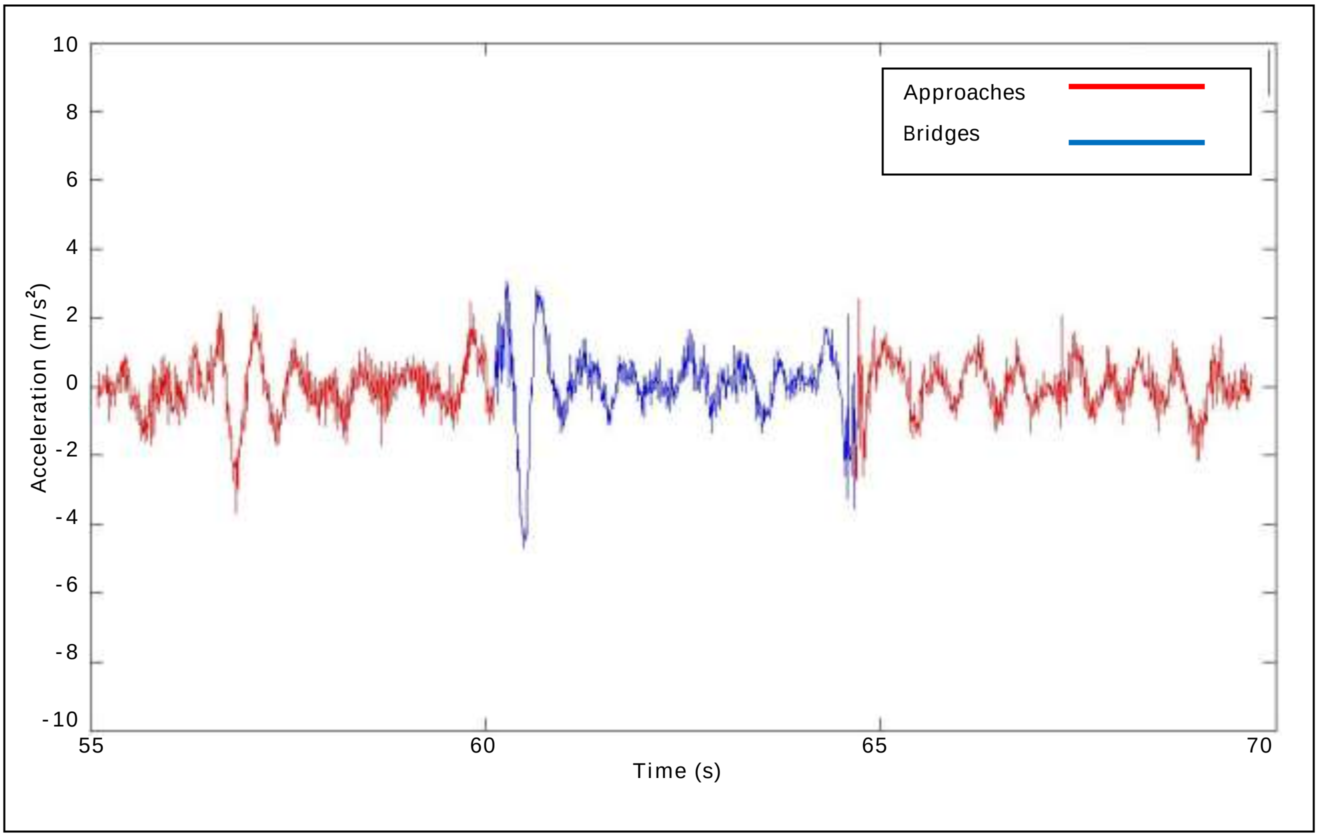


Figure 4.12 Driveover response (70km/h) at Waikanae bridge (southbound – increasing direction)

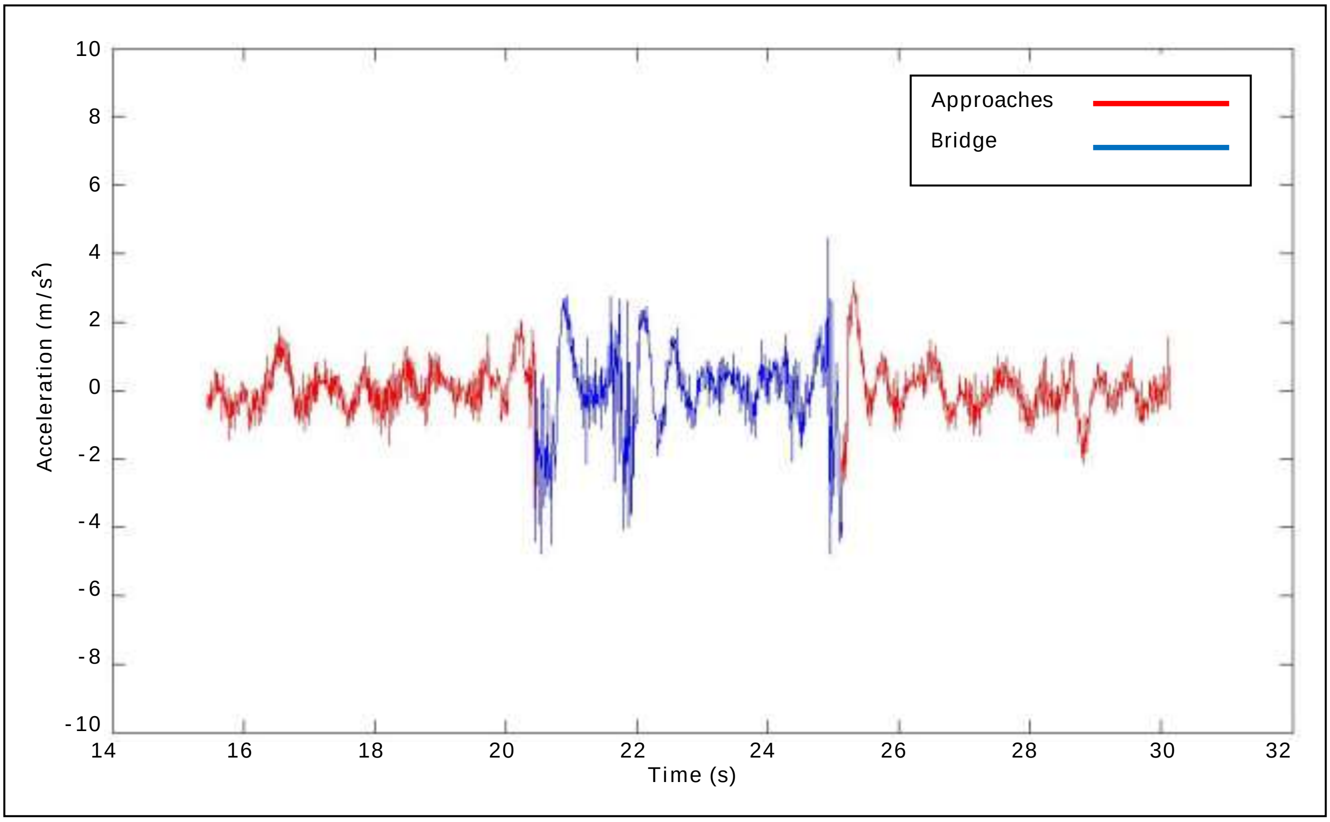


Figure 4.13 Driveover response (100km/h) at Manawatu bridge (Whirokino) (northbound – decreasing direction)

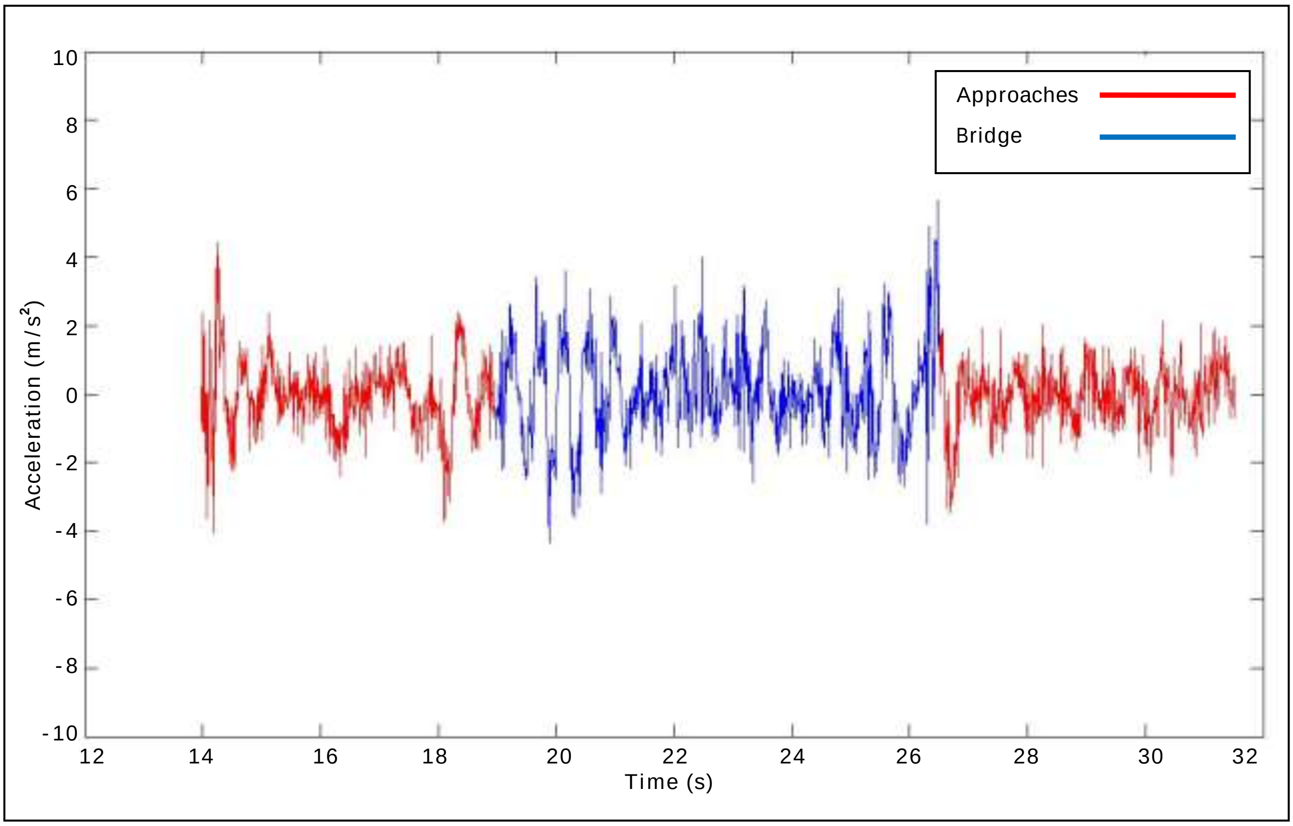


Figure 4.14 Driveover response (100km/h) at Manawatu bridge (Whirokino) (southbound – increasing direction)

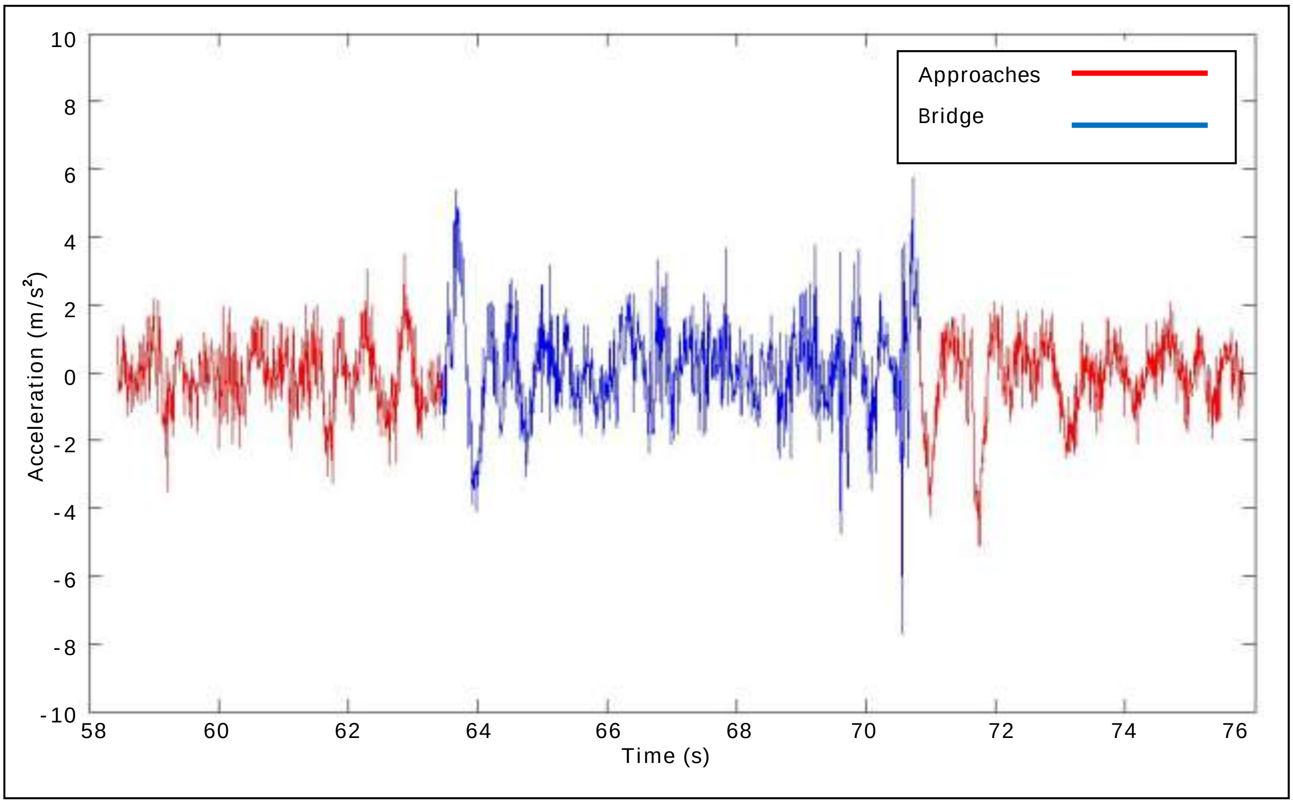
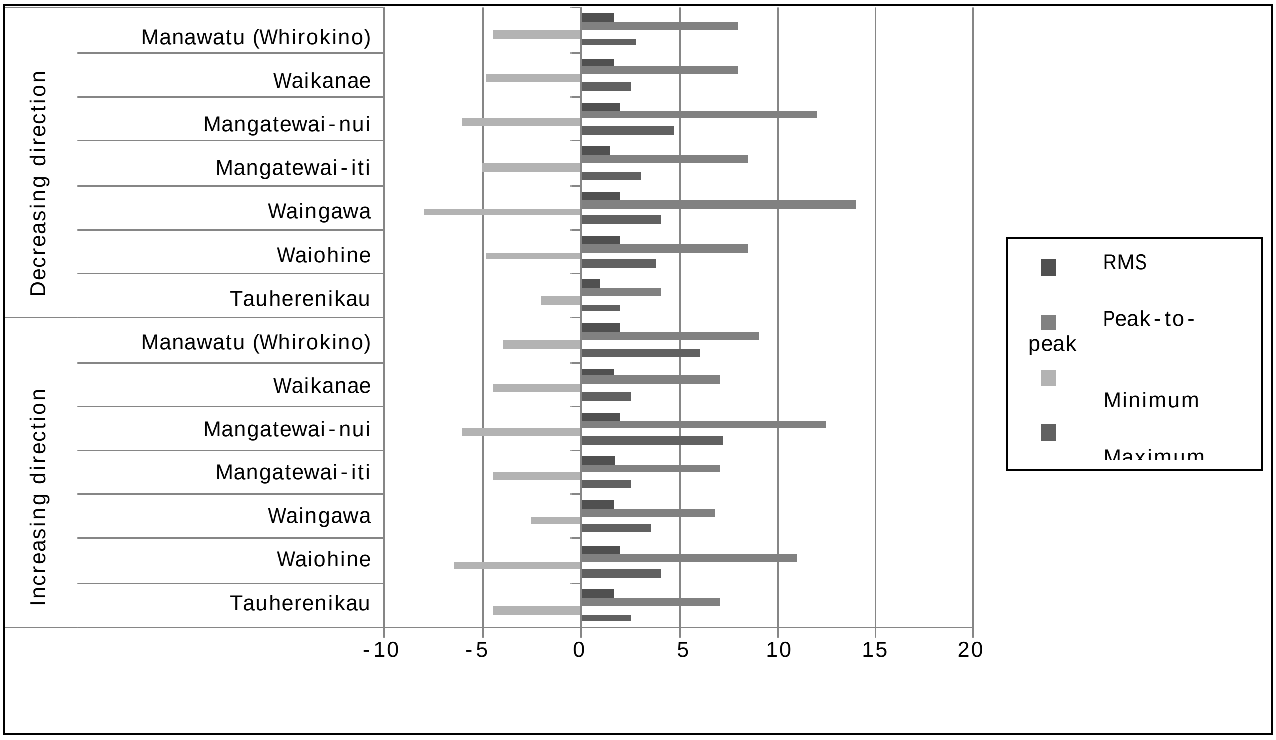


Table 4.1 Measured accelerations driving onto the bridge

Bridge	Maximum positive (up) (m/s^2)	Maximum negative (down) (m/s^2)	Peak-to-peak (m/s^2)	RMS
Tauherenikau – increasing	2.66	-4.54	7.20	1.13
Tauherenikau – decreasing	2.12	-2.25	4.37	0.85
Waiohine – increasing	4.49	-6.07	10.56	1.52
Waiohine – decreasing	3.70	-4.70	8.41	1.62
Waingawa – increasing*	3.69	-2.80	6.49	1.07
Waingawa - decreasing	4.26	-8.90	13.16	1.74
Mangatewai-iti – increasing	2.70	-4.57	7.27	1.29
Mangatewai-iti – decreasing	3.23	-5.17	8.41	1.33
Mangatewai-Nui – increasing	6.90	-5.60	12.50	1.54
Mangatewai-Nui – decreasing	4.96	-5.72	10.68	1.66
Waikanae – increasing*	2.76	-4.76	7.52	1.33
Waikanae – decreasing*	3.07	-4.69	7.75	1.45
Manawatu (Whirokino) – increasing	5.38	-4.08	9.46	1.74
Manawatu (Whirokino) – decreasing	3.41	-4.33	7.74	1.46

*(70km/h)

Figure 4.15 Driveover peak-to-peak accelerations on bridge approaches



The following observations can be made from the driveover response plots shown in figures 4.1 to 4.14, and from the measured driveover acceleration levels listed in table 4.1:

- The acceleration levels at both ends of the bridges are typically higher than those either on the central sections of the bridges or on the adjacent road sections.
- The vehicle's responses to each of the separate bridge spans can be clearly seen in most of the acceleration traces.
- Peak-to-peak acceleration levels vary from 4.37m/s^2 to 13.16m/s^2 (a factor of three) across the seven bridges and fourteen approaches.

5. Measured dynamic bridge responses

5.1 Background, setup and testing

Following the recommendations of the steering group to monitor the accelerations at the centre of each first bridge span under freestream traffic conditions, each of the seven bridges chosen was instrumented with a triaxial accelerometer to measure the vertical acceleration levels. A three-axis Syflex™ accelerometer was placed at the centre of the first bridge span at the side of the bridge parapet or walkway, as seen in figure 5.1.

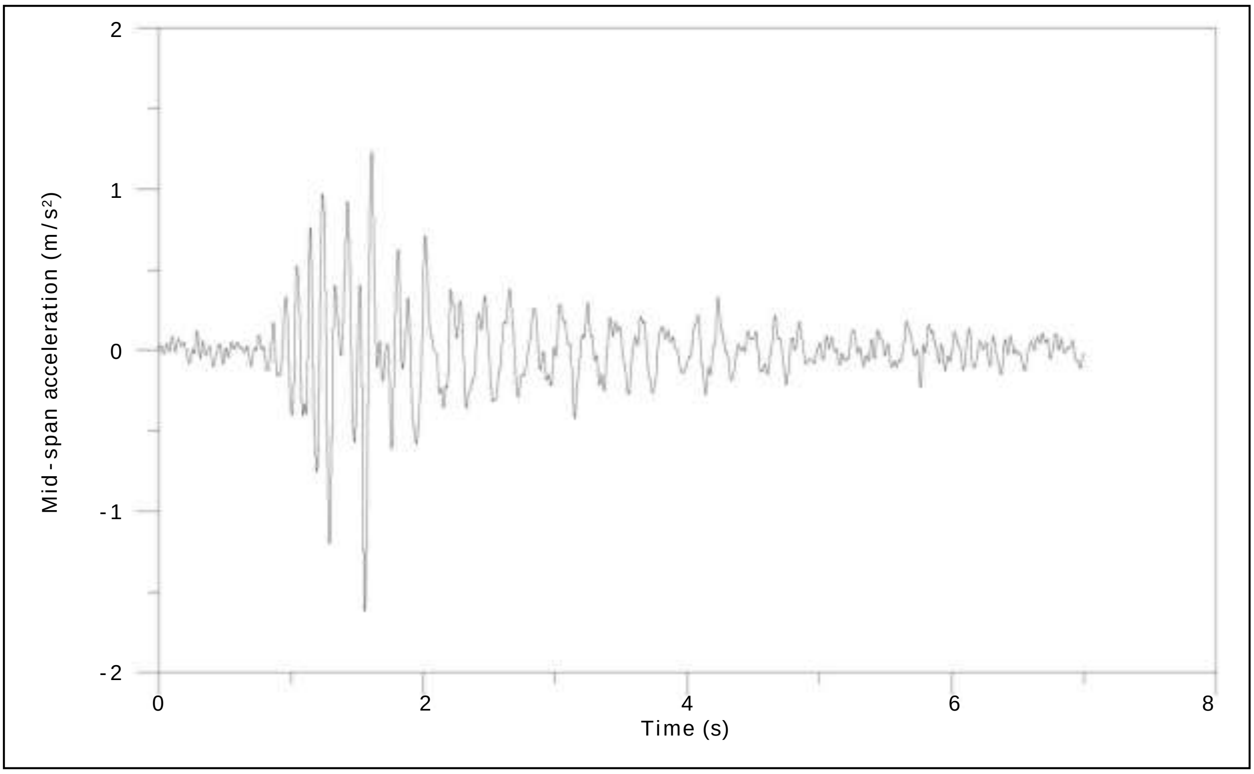
Figure 5.1 Syflex™ triaxial accelerometer on bridge edge (centre of first span)

The accelerometer was connected to a PC-based data acquisition system, and the acceleration levels were recorded in one continuous data file at 1000Hz as vehicles travelled across the bridge span. An event marker was used to identify different classes of vehicles, these being cars, light to medium commercial vehicles and HCVs. Measurements were carried out at both ends of the bridges until between 25 and 30 passes by HCVs had been recorded. Note that the 70km/h speed limit close to the Waingawa bridge and the 70km/h speed limit on the Waikanae bridge would suggest that the speed regimes for HCVs on these bridges would be less than on the other five bridges, where open road speed limits apply.

5.2 Data processing and analysis

To extract the bridge response data for the HCVs only, a MATLAB software routine was written to generate individual data files for each HCV pass. Figure 5.2 shows an acceleration time history for one of the individual HCV passes.

Figure 5.2 Sample mid-span acceleration–time history for a single HCV pass



This shows an increase in the acceleration level over a number of cycles, followed by a gradual decay. Another software routine was developed to do a forward and backward double integration, first to provide velocity and then displacement. Figures 5.3 to 5.5 show the acceleration, velocity and displacement time histories for the record shown in figure 5.2.

Figure 5.3 Mid-span acceleration history for a single HCV pass

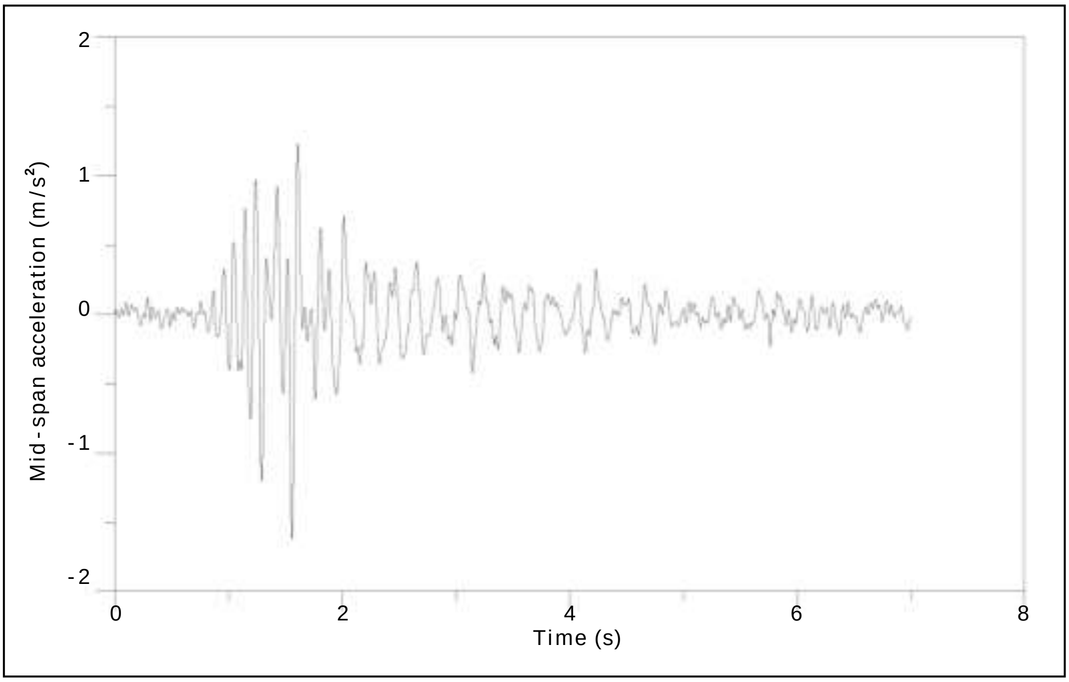


Figure 5.4 Mid-span velocity history for a single HCV pass

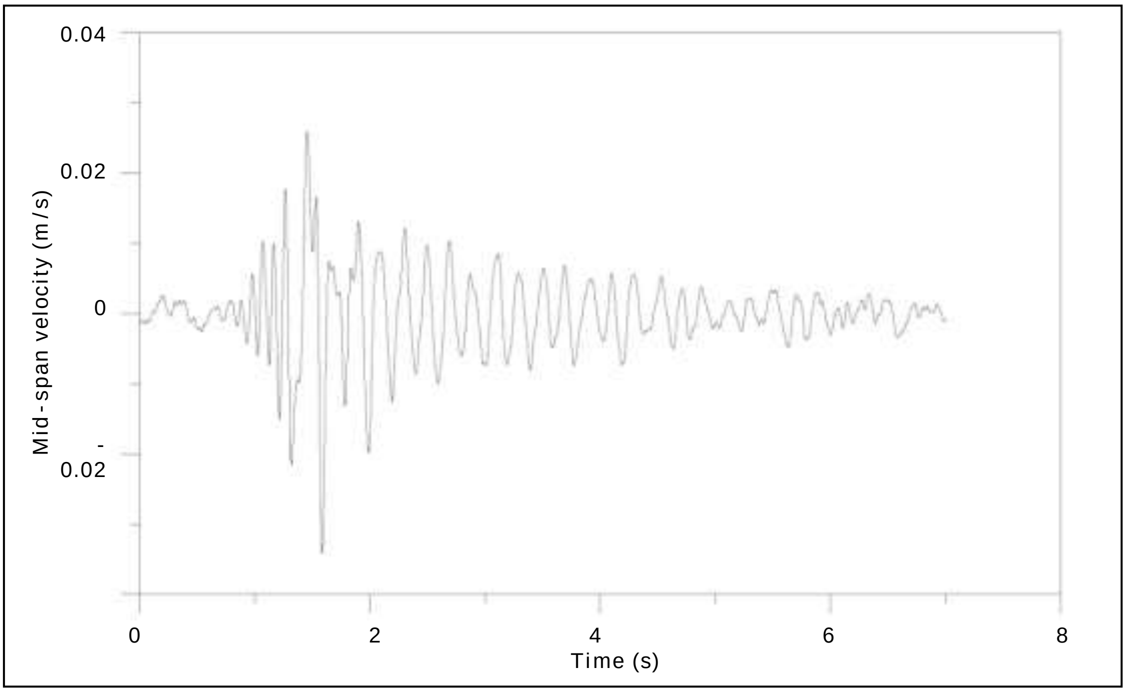
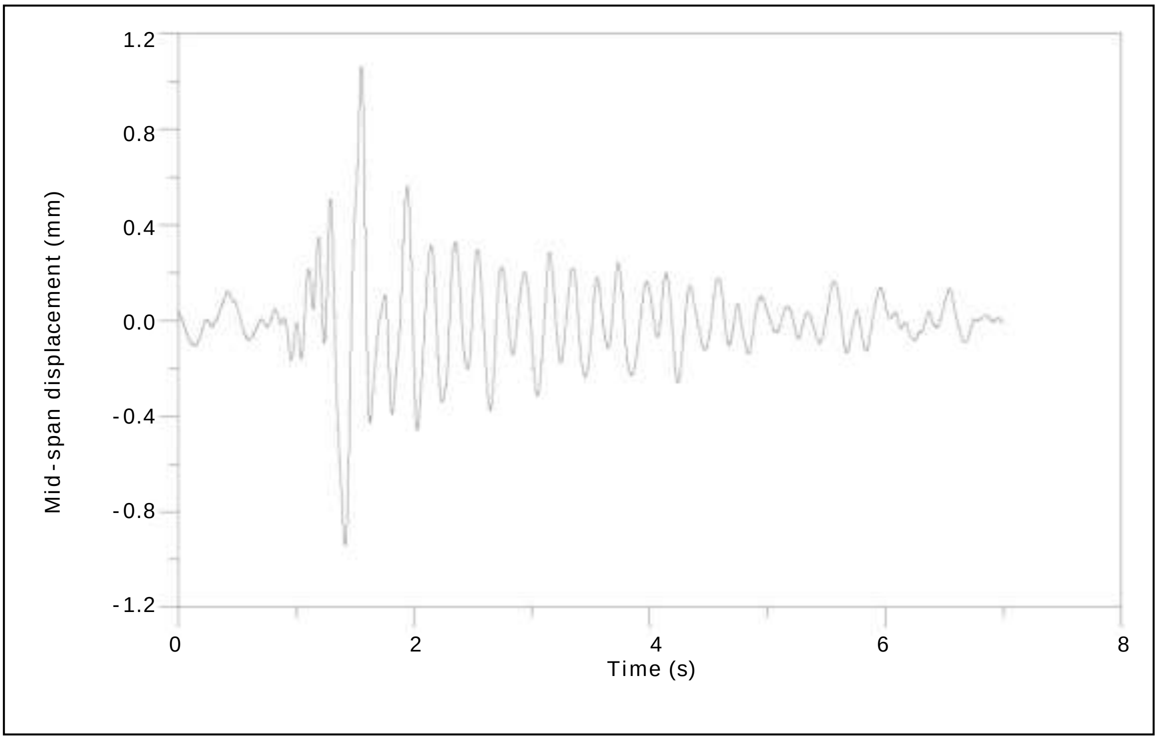


Figure 5.5 Mid-span displacement history for a single HCV pass



Each of the individual HCV passes was processed in this way, and then the peak-to-peak maximum displacements were calculated. Table 5.1 lists the average and maximum peak-to-peak displacements for each end of the seven bridges investigated.

Table 5.1 Average and maximum peak-to-peak vertical displacements

Bridge	Average peak-to-peak displacement (mm)		Maximum peak-to-peak displacement (mm)	
	Increasing	Decreasing	Increasing	Decreasing
Tauherenikau	1.02	0.91	1.38	1.32
Waiohine	0.87	1.16	2.01	2.39
Waingawa[a]	1.18	1.43	1.95	2.41
Mangatewai-iti[b]	0.77	0.88	1.64	1.97
Mangatewai-nui	2.18	2.62	4.93	5.58
Waikanae[a]	0.67	0.57	0.71	0.68
Manawatu (Whirokino)	1.25	1.49	1.95	2.02

Notes to table 5.1:

a Speed regime is likely to be less than 90km/h.

b Approaches were smoothed prior to testing

6. Analysis of data

6.1 Structure

The data analysis is presented in two sections, the first being a comparison of the profile variance data with the measured driveover data; the second is a comparison of the profile variance data and the measured bridge response data.

6.2 Profile variance and driveover accelerations

Figures 6.1 to 6.3 compare the profile variance data for the three wavelengths (3m, 10m and 30m) with the measured peak-to-peak driveover response data. Note that the profile variance data has been averaged from the left and right wheelpath values. It is important to remember here that the driveover tests for Waikanae (both directions) and Waingawa (increasing direction) were carried out at 70km/h, and the Manatewai-iti bridge approaches were smoothed prior to testing. Accordingly, the Waikanae and Waingawa peak-to-peak driveover accelerations could be expected to be higher if they were done at 100km/h. Similarly, the profile variance data for Mangatewai-iti should be lower than that recorded in the RAMM database, assuming that the smoothing works were successful.

Figure 6.1 Comparison of 3m profile variance and peak-to-peak driveover accelerations

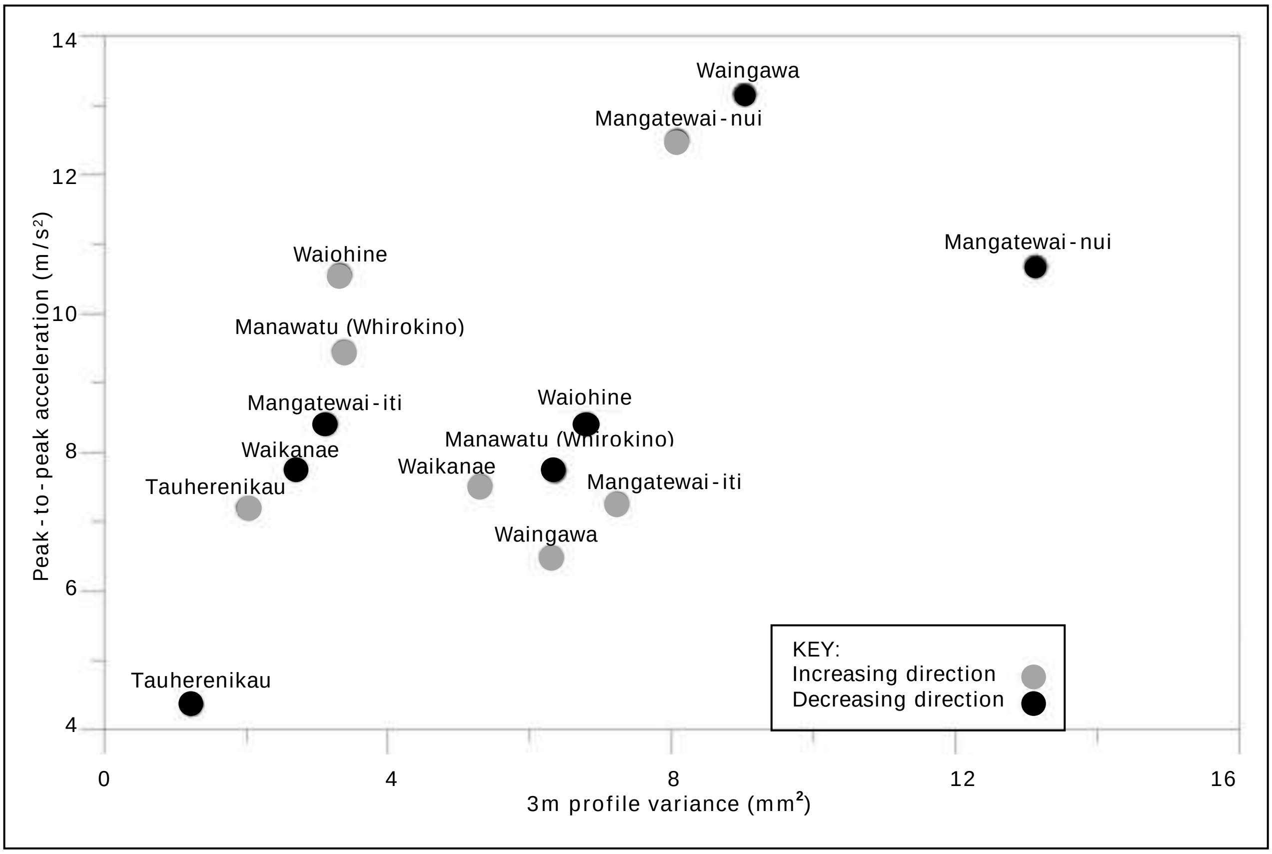


Figure 6.2 Comparison of 10m profile variance and peak-to-peak driveover accelerations

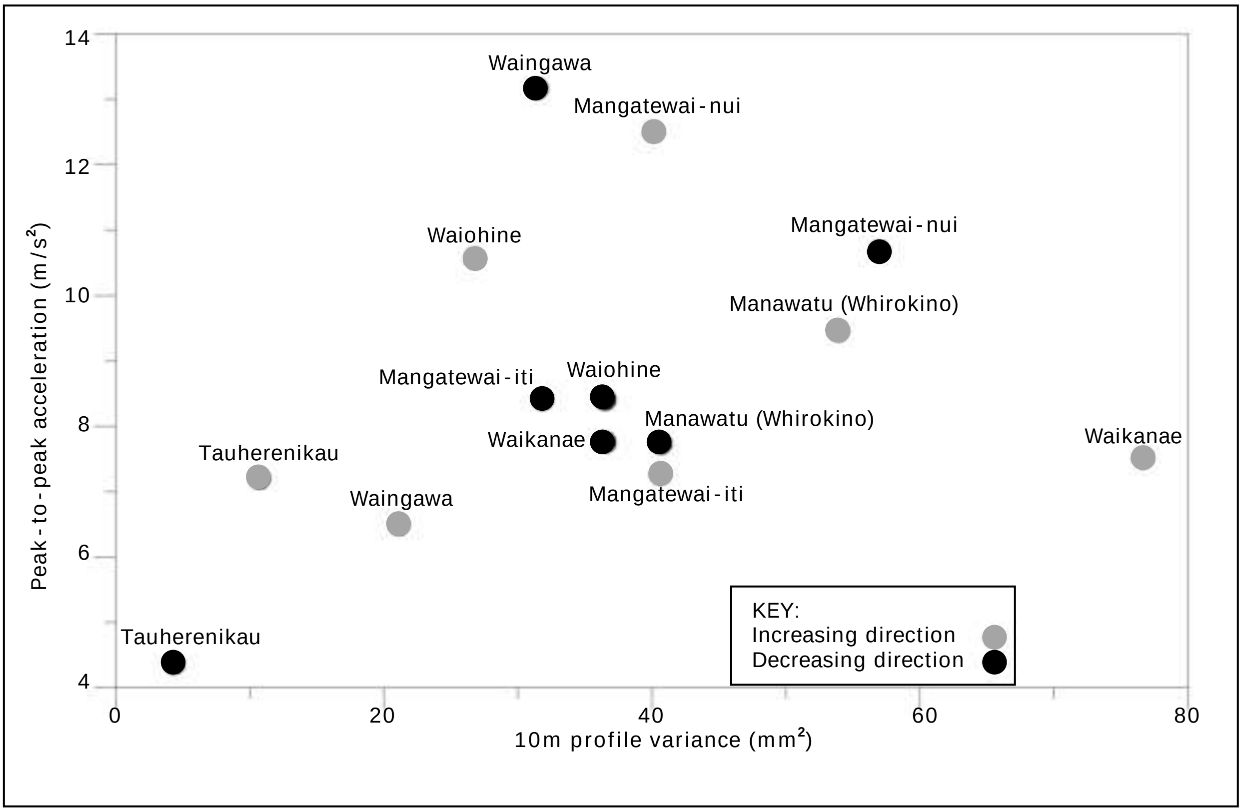
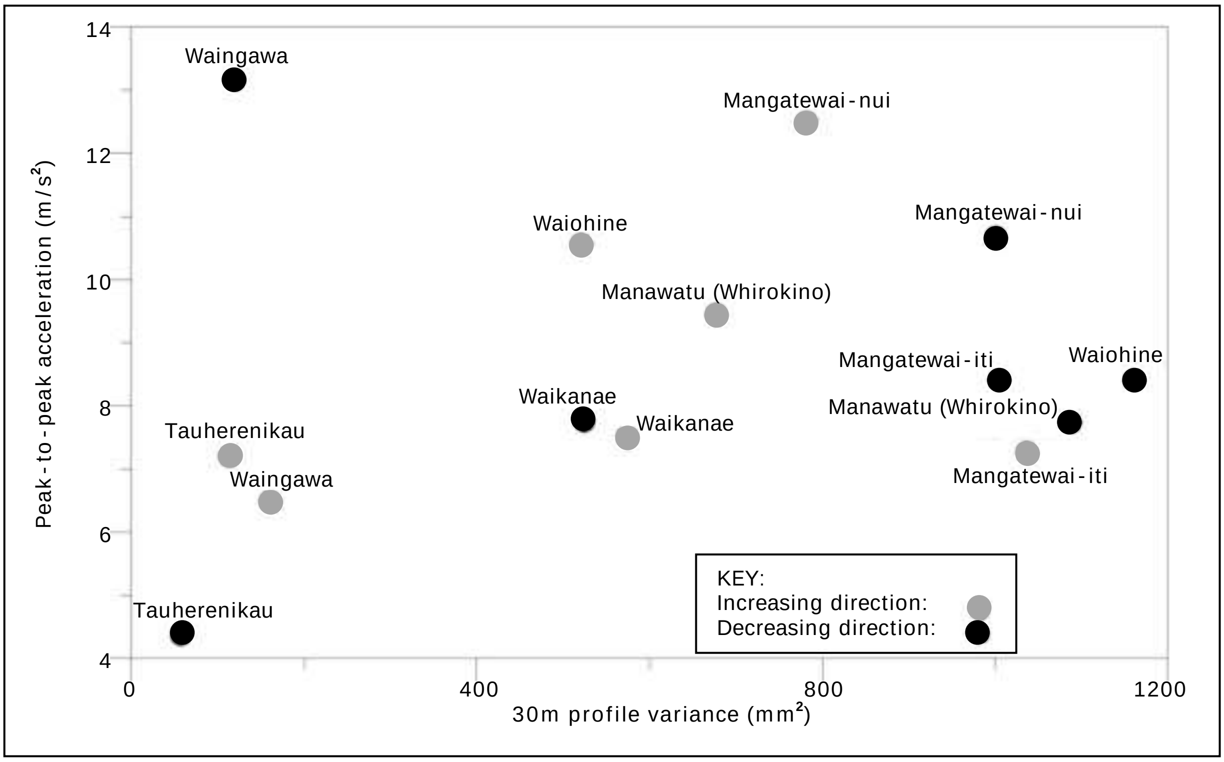


Figure 6.3 Comparison of 30m profile variance and peak-to-peak driveover accelerations



The peak-to-peak accelerations measured during the driveover tests generally increase with increasing profile variance across all three wavelengths, although considerable scatter appears in the data, more so for the 10m and 30m wavelengths. It may be that the response of the vehicle chosen is not sufficiently similar to that of an HCV.

6.3 Profile variance and bridge response

Figures 6.4 to 6.6 compare the profile variance for the three wavelengths (3m, 10m and 30m) with the average peak-to-peak displacements of the first bridge spans. The profile variance data has been averaged from the left and right wheelpath values. It is important to remember here that the speeds of the trucks passing over the Waikanae bridge (both directions) and the Waingawa bridge (increasing direction) are likely to be lower than the open road speed limit, given the nearby proximity of signed 70km/h speed zones. In addition, the smoothing works carried out on the Manatewai-iti bridge approaches prior to testing would suggest that the profile variance data for this bridge should be lower than that recorded in the RAMM database, assuming that the smoothing works were successful.

It was assumed that the structures of both ends of each bridge would be similar, but it is known that the profile variance values at both ends are not. The differences in the measured displacements at each end of a particular bridge should be a result of the different effects on the bridges generated by the test vehicle's responses to the approaches and abutments. Accordingly, the data points for both ends of each bridge have been connected to assess this relationship.

Figure 6.4 Comparison of 3m profile variance and average peak-to-peak bridge displacement

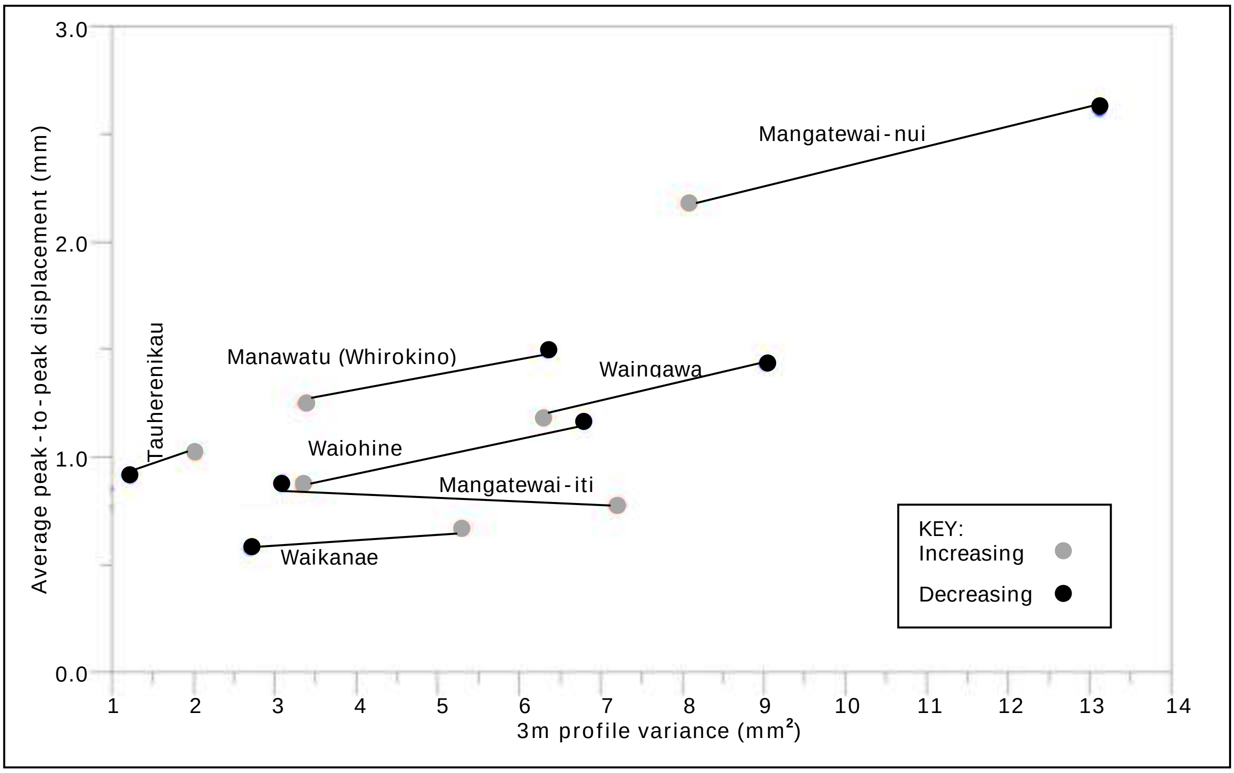


Figure 6.5 Comparison of 10m profile variance and average peak-to-peak bridge displacement

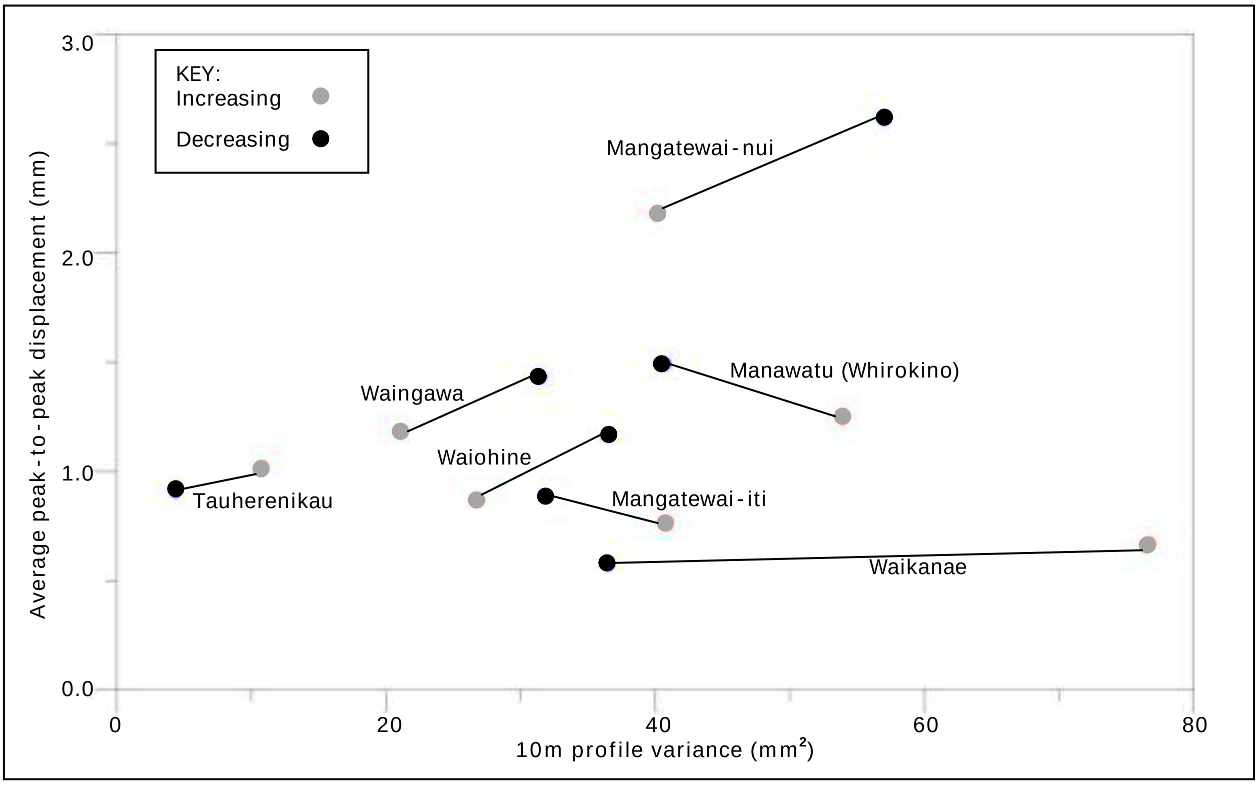
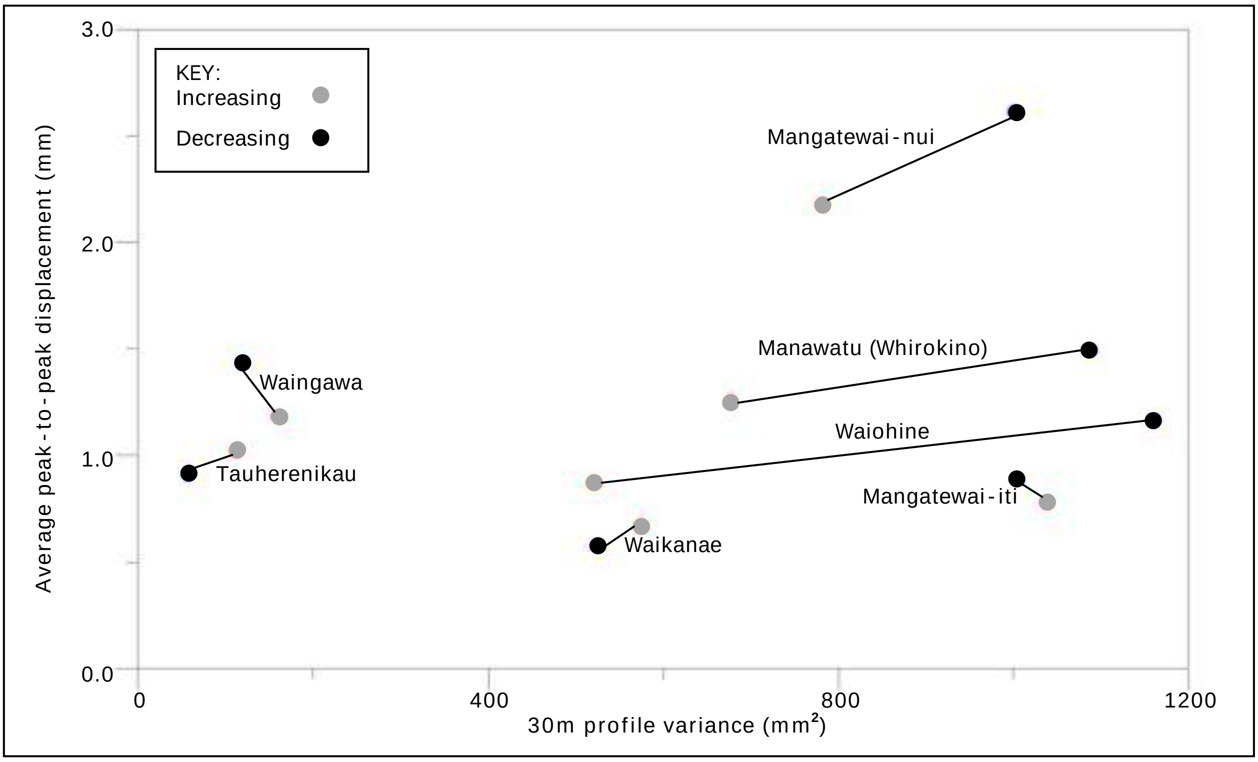


Figure 6.6 Comparison of 30m profile variance and average peak-to-peak bridge displacement



The average peak-to-peak displacements show generally consistent increases with increasing 3m profile variance (figure 6.4), given that (1) the correct profile variance at the Mangatewai-iti bridge is unknown, and (2) the speeds over the Waikanae and possibly the Waingawa bridges in the increasing direction are likely to be higher than that in the decreasing direction because of the proximity of posted speed limit changes. The trends are mostly similar for the 10m profile variance (figure 6.5), apart from the Whirokino bridge. With the 30m profile variance (figure 6.6), most of the bridges show the same trends, but with Waingawa, the trend is reversed. However, the profile variance differences in the increasing and decreasing directions for this site are relatively small.

To further assess the variation of the average peak-to-peak displacement with profile variance for each of the three wavelengths, table 6.1 lists the differences in displacement divided by the differences in profile variance.

Table 6.1 Change in average peak-to-peak bridge displacement with profile variance

Bridge	**Δ** displacement (mm)/**Δ** profile variance (mm^2)		
	3m wavelength	10m wavelength	30m wavelength
Tauherenikau	0.137	0.017	0.0020
Waiohine	0.084	0.030	0.0005
Waingawa[a]	0.092	0.024	-0.0062
Mangatewai-iti[b]	-0.027	-0.013	-0.0034
Mangatewai-nui	0.086	0.026	0.0020
Waikanae[a]	0.036	0.002	0.0019
Manawatu (Whirokino)	0.081	-0.018	0.0006
Average[c]	0.086	0.014	0.00012

Notes to table 6.1:

a Speed regime likely to be less than 100km/h.

b Approaches smoothed prior to testing.

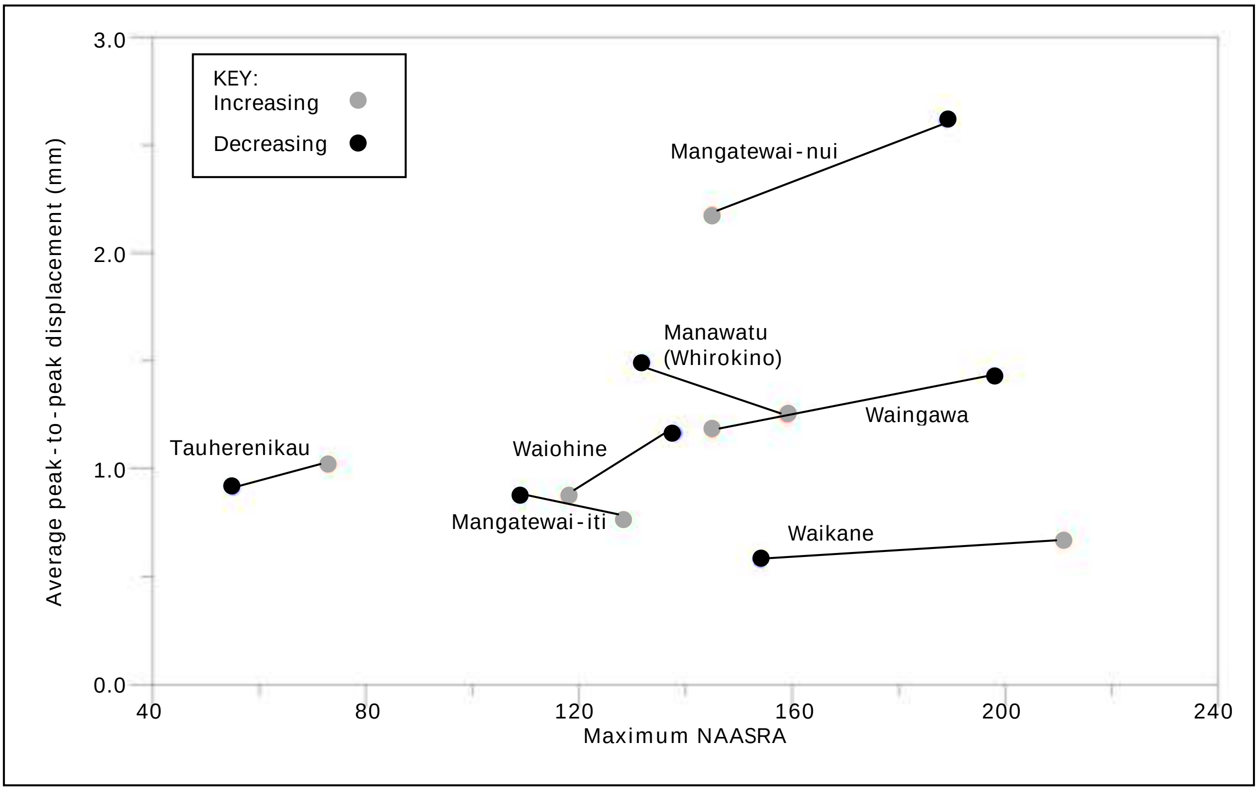
c Excludes Mangtewai-iti because of uncertain profile variance data.

This shows that on each bridge (excluding Managtewai-iti), the displacement consistently increases with an increase in profile variance, and that the levels of these increases are generally similar for the 3m wavelength. The trends for the 10m and 30m wavelength are not as consistent.

6.4 NAASRA roughness and bridge response

Figure 6.7 compares one of the other measures of surface roughness, the NAASRA roughness values, with the average peak-to-peak displacements of the first bridge spans. In contrast to the profile variance, the NAASRA roughness does not provide any information on the wavelength content of the roughness measurement.

Figure 6.7 Comparison of NAASRA roughness and average peak-to-peak displacement



This plot shows trends that are more similar to those for the medium 10m wavelength profile variance than either the short 3m or long 30m wavelengths. It suggests that the NAASRA roughness is not the most appropriate roughness parameter to use as a screening tool for dynamic bridge loads.

7. Profile variance as a screening tool

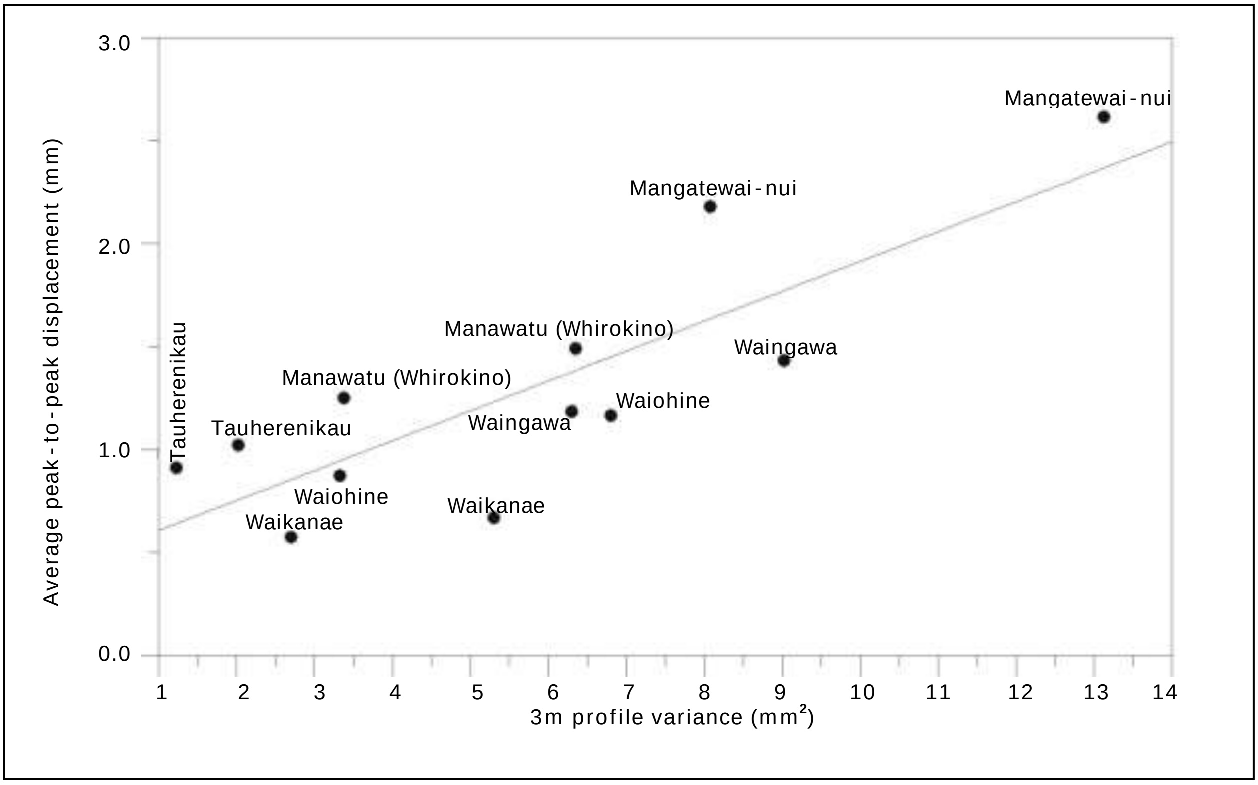
7.1 Discussion

It has been shown that a strong relationship exists between the measured bridge peak-to-peak displacements and the 3m wavelength profile variance data, irrespective of the bridge characteristics, including the type, materials and span length. Replotting figure 6.4 as figure 7.1 with a best fit linear regression added, and the distinction between increasing and decreasing directions removed, shows a correlation coefficient of almost 70%. This is a strong correlation, given the potential uncertainties in the speeds and vehicle loads passing over the different bridges.

It has also been shown (figure 6.4) that for the individual bridges, where the structures at each end can be assumed to be similar, the differences in the 3m profile variance from one end to the other show generally consistent increases in the average peak-to-peak first span displacements. This indicates that the 3m profile variance can be used to predict the first span bridge responses, and hence give a relative measure of dynamic first span bridge loads.

The relationship between the profile variance and displacement data does not give a measure of the magnitude at which the displacement starts to cause damage to bridges. We have shown that for freestream HCVs, the first span bridge displacement increases with increasing 3m profile variance. While we know that the dynamic loads on bridges will generally increase with vehicle weight and speed, we do not know the specific relationships between profile variance, bridge displacement, vehicle load and speed.

Figure 7.1 Linear regression of average peak-to-peak displacement and 3m profile variance



Notes to figure 7.1:

a Mangatewai-iti has been removed, given the uncertainty in the profile variance data created by the smoothing works undertaken at this bridge.

b Regression equation: displacement = 0.145 x 3m profile variance = 0.463

c Coefficient of determination: R^2 = 0.6852, residual mean square

It seems reasonable to take the UK Roads Board (2003) 3m wavelength profile variance threshold for poor ride quality of 10mm^2, described in section 2.2, as a basis for initial selection of bridges likely to suffer from high dynamic loads. It can be seen from figure 6.8 that only the Mangatewai-nui bridge and one end of the Waingawa bridges approach or surpass this value.

We know that a level of 3m profile variance over 10mm^2 will produce poor ride quality for commercial vehicles (Jamieson 2008). In general, roughness can be improved through smoothing works. Smoothing works were carried out on the Mangatewai-iti bridge approaches prior to the on-road testing described earlier. However, we do not know whether such smoothing has improved the 3m, 10m or 30m wavelength profile variance values, or all three. Some indication will be known when the data from the 2008/09 and 2009/10 annual network surveys can be compared for the Mangatewai-iti bridge.

As the NZTA is also tasked with providing good ride quality on roads and bridges, as well as good quality bridge asset management, the ability to identify poor ride quality issues and less than desirable dynamic loading on bridges, even if the overall impact on structural performance may be minor, is also important. The profile variance data, particularly for the 3m wavelength, provides the ability to select and prioritise bridges and approaches for treatment to improve ride quality.

7.2 Suggestions for further work

The findings of the bridge response measurements suggest a number of avenues for further work to identify and quantify the effects of the 3m wavelength profile variance on the dynamic bridge loads. These could include:

- carrying out a national screening of bridges to identify bridges with 3m wavelength profile variance levels over 10mm^2 that are potentially exposed to high dynamic loads, and encourage regional and network bridge consultants to assess smoothing options
- extending the bridge displacement measurements to cover a wider range of bridge types, sizes, profile variance values and traffic loadings
- finding out if the smoothing works on the Mangatewai-iti bridge approaches actually reduced the peak 3m profile variance value, by comparing the 2008/09 network survey values with those from the 2009/10 survey
- selecting one or two bridges with peak 3m profile variance values on the approaches/abutments in excess of the 10mm^2 threshold for poor ride quality as candidates for smoothing work
- measuring the first span's static and dynamic displacements on these bridges using an HCV and varying both its load and speed, up to the maximum acceptable limits, including the proposed new weight limits
- carrying out smoothing works on the bridges
- repeating the dynamic displacement measurements using the same combination of vehicle, load and speed
- identifying the changes in the profile variance levels resulting from the smoothing works

- quantifying the effects of any changes in the 3m profile variance on the dynamic displacements and dynamic impact factors, including any variation with vehicle speed and load.

8. Conclusions and recommendations

8.1 Overview

The following conclusions are drawn from a comparison of measured dynamic bridge deflections on selected bridges on the state highway network with road profile variance data and roughness, and geometry data from the NZTA's RAMM database. Recommendations for additional work are also made.

8.2 Conclusions on the profile variance screening process

8.2.1 Profile variance of bridges and approaches: current status

Comparison of the RAMM data for bridges and their approaches within 100m of the bridge abutments shows the following:

- NAASRA roughness on bridges and bridge approaches is, on average, around 35% higher than on the adjacent roads, up from around 60 NAASRA counts/km to around 80.
- Short wavelength (3m) profile variance on bridges and bridge approaches is, on average, around 60% higher than on the adjacent roads, up from around 1.2mm^2 to around 2mm^2.
- Medium wavelength (10m) profile variance on bridges and bridge approaches is, on average, around 100% higher than on the adjacent roads, up from around 7mm^2 to around 14mm^2.
- Long wavelength (30m) profile variance on bridges and bridge approaches is, on average, around 90% higher than on the adjacent roads, up from around 135mm^2 to around 260mm^2.
- Maximum values of short, medium and long wavelength profile variance on bridges and approaches across the state highway network often exceed the thresholds for poor ride quality used by the UK Roads Board (2003). These maximum values on bridges and their approaches most often occur around the bridge abutments and bridge deck joints.

8.2.2 Vehicle driveover accelerations

Measurements of an instrumented vehicle's response to the approaches, abutments and decks of selected bridges showed the following:

- The vertical accelerations were highest in the immediate vicinity of the bridge abutments, although peaks were also observed around the bridge deck joints.
- Maximum peak-to-peak vertical accelerations on the first bridge spans ranged from around 4.4m/s^2 to 13.2m/s^2.

8.2.3 Bridge deflections

Measurements of the dynamic deflections of the first bridge spans caused by HCVs in the freestream traffic showed that maximum peak-to-peak dynamic deflections on the first bridge spans ranged from around 0.7mm to 5.6mm for HCVs. Average peak-to-peak dynamic deflections on the first bridge spans ranged from around 0.6mm to 2.6mm.

8.2.4 Comparison of bridge deflections and profile variance

Comparison of the measured dynamic bridge deflections of the first bridge spans and the measured values of profile variance for short, medium and long wavelengths revealed the following:

- The bridge deflections, both the maximum and average peak-to-peak values, generally increased with increasing profile variance across the short (3m), medium (10m) and long (30m) wavelengths.
- The most significant predictor of the dynamic bridge displacements, and hence the dynamic loads, was the short (3m) wavelength profile variance, with a correlation coefficient of around 70%, even allowing for expected variations in traffic loads and speeds.
- Comparing the different levels of profile variance at both ends of each bridge that was assumed to have the same structure at each end showed consistent increases of the dynamic displacements with the 3m profile variance.
- The 3m wavelength profile variance can be used to rank bridge approaches and abutments for dynamic loads. Using the UK Roads Board (2003) threshold of $10mm^2$ for poor truck ride quality, the 3m profile variance can also potentially be used to screen bridges for high dynamic loading, and to identify bridges where maintenance or smoothing works could substantially reduce these dynamic loads and improve bridge life.

8.3 Recommendations

The recommendations for further work arising from this study of the effects of profile variance on the dynamic response of bridges are as follows:

- Carry out a national screening of bridges to identify bridges with 3m wavelength profile variance levels over $10mm^2$ that are potentially exposed to high dynamic loads, and encourage regional and network bridge consultants to assess smoothing options.
- Extend the bridge displacement measurements to cover a wider range of bridge types, sizes, profile variance values and traffic loadings.
- Determine whether the smoothing works on the Mangatewai-iti bridge approaches reduced the peak 3m profile variance values, by comparing the 2008/09 network survey values with those from the 2009/10 survey.
- Select one or two bridges with peak 3m profile variance values on the approaches/abutments in excess of the UK Roads Board (2003) $10mm^2$ threshold for poor ride quality as candidates for smoothing work.
- Measure the first span static and dynamic displacements on these bridges using an HCV, varying both its load and speed, up to the maximum acceptable limits, including the proposed new weight limits.
- Carry out smoothing works on the bridges.
- Repeat the dynamic displacement measurements using the same combinations of vehicle, load and speed.
- Identify the changes in the profile variance levels resulting from the smoothing works.
- Quantify the effects of any changes in 3m profile variance on the dynamic displacements and dynamic impact factors, including any variation with vehicle speed and load.

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Appendix Calculating longitudinal profile variance

The calculation of longitudinal profile variance (UK Roads Board 2003) is carried out as follows:

First, the number of profile points (*m*) corresponding to a moving average length (eg 3m, 10m or 30m) is calculated using equation A1:

$$m = \frac{L_{ma}}{l} \text{ (rounded to the nearest odd integer)} \qquad \text{(Equation A1)}$$

Where:

- L_{ma} = the moving average length (e.g. 3m, 10m or 30m)
- l = interval between profile point readings (eg 0.1m)

Thus, using equation A1, for 3m, 10m and 30m moving average lengths with a reading interval of 0.1m, the number of points would be 31, 101 and 301 respectively.

The number of profile points corresponding to the length *L* over which the longitudinal profile variance is to be averaged (eg 10m) is calculated using equation A2:

$$J = \frac{L}{l} \text{ (rounded down to the nearest integer)} \qquad \text{(Equation A2)}$$

Where:

- J = the number of profile points corresponding to the length over which the longitudinal profile is to be averaged
- l = interval between profile point readings (eg 0.1m)

Thus, using equation A2, for 10m averaging lengths with a reading interval of 0.1m, the number of points would be 100.

For each point *k* on the survey run, a moving average (*i*) must be calculated. However, first, the range of *k* must be determined using equation A3:

$$k_{\min} = \left(\frac{m+1}{2}\right)$$

$$k_{\max} = \left(\frac{M - m - 1}{2}\right) \qquad \text{(Equation A3)}$$

Where:

- M is the total number of readings in the run.

The moving average for each point *k* is calculated using equation A4:

$$i = k - \frac{m-1}{2} \qquad \text{(Equation A4)}$$

The profile amplitude, *Y*, is calculated for each point *k* using equation A5:

$$\overline{Y_k} = \left(\frac{1}{m}\right) \sum_{k=i}^{k=i+m-1} Y_j \qquad \text{(Equation A5)}$$

For each point k on the survey run, a profile amplitude deviation (d_k) from its corresponding moving average is calculated using equation A6:

$$d_k = Y_k - \overline{Y_k} \qquad \text{(Equation A6)}$$

Where:

- Y_k = profile amplitude at point k.

The moving average longitudinal profile variance (LPV) over each length L starting at point i is then calculated using equation A7:

$$LPV_i = \left(\frac{10^6}{J}\right) \sum_{k=i}^{i+J-1} (d_k)^2 \qquad \text{(Equation A7)}$$

The 10^6 factor is used to convert from m^2 to mm^2.