DYNAMIC LOAD EFFECTS OF HEAVY VEHICLES ON NEW ZEALAND HIGHWAY BRIDGES

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CONTENTS

EXECUTIVE SUMMARY 7

ABSTRACT 8

1. INTRODUCTION 9
   1.1 General 9
   1.2 Objectives 10
   1.3 Overall Procedure 10

2. INSTRUMENTATION 11
   2.1 Survey of Logging Systems Available 11
   2.2 Development of Data Logger 11

3. FIELD WORK 12
   3.1 Selection of Bridges 12
   3.2 Installation of Recording Equipment 12
   3.3 Calibration of Bridge and Equipment 13
   3.4 Acquisition of Data 13

4. ANALYSIS OF RESULTS FROM MAIN BEAMS 14
   4.1 Initial Reduction of Raw Data 14
   4.2 Processing of Raw Data 15
   4.3 Parameters Considered in the Analysis 16
   4.4 Individual Bridge Data Analysis 16
   4.5 Correlation of Individual Bridge Data Analyses 17

5. DISCUSSIONS AND CONCLUSIONS FROM MAIN BEAM RESULTS 19
   5.1 Individual Bridge Analyses 19
   5.2 Correlation Between Bridges at Fixed Load Intensity 19
   5.3 Correlation Between Bridges at All Load Intensities 20
   5.4 Conclusions from Main Member Results 20

6. DISCUSSION AND CONCLUSIONS FROM DECK SLAB RESULTS 22
   6.1 Procedures Followed 22
   6.2 Discussion and Conclusions from Deck Slab Results 22

7. RECOMMENDATIONS 24

TABLES AND FIGURES 25-58

REFERENCES 59

Appendix A Survey of Available Recording Equipment A-60-66

Appendix B Description of Recording Equipment Developed A-67-69

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The manager for this project was H E Chapman of *Works Consultancy Services*.

The recording equipment was designed and built, and Appendices A and B were written, by J A Gould of *Works Central Laboratories*. The equipment was installed in the field by G A Beattie of *Works Central Laboratories*.

NOTE

Works Consultancy Services Ltd and Works Central Laboratories are now part of Opus International Consultants and trade under that name.

EXECUTIVE SUMMARY

The formula for Impact Factor on structural members given in the Transit New Zealand Bridge Manual uses span length as the only parameter.

Because Impact is a complex effect, dependent on various properties of the span and the traffic it is carrying, the simplified nature of the formula produces a generally conservative value. This project aimed to verify the appropriateness of the formula and to propose an alternative if warranted.

Instrumentation was developed and deployed on 12 bridges with spans ranging from 9 to 30 metres. Impact Factors were recorded under normal traffic conditions and more than 22 000 loading events were recorded.

The correlation of Impact Factors observed on main beams and deck slabs was checked against bridge parameters of span length, natural frequency, damping, support member material, end conditions and road roughness. Vehicle parameters checked were the proportion of design load and vehicle speed.

The only significant correlation was with proportion of design load, although there was marked variation between bridges.

The conclusion was that although there was wide variation in the Impact experienced by different bridges overall, the variation at design load level was limited, and that current design criteria covered this adequately, and should be retained at the present time.

ABSTRACT

Impact Factors were recorded on main members and deck slabs (where appropriate) of 12 bridges of between 9 and 30 metre span, under normal traffic conditions. Correlation of Impact Factor was checked against span length, natural frequency, damping, support member material, end conditions and road roughness, and against vehicle speed and proportion of design load. Significant correlation was seen only against proportion of design load. Although there was marked variation between the bridges, the variation at design load level was limited. The conclusion was that current design criteria covered the Impact Factor adequately and should be retained.

1 INTRODUCTION

1.1 General

This report describes work carried out by Works Consultancy Services Limited for Transit New Zealand. The period of the project was from June 1992 to June 1994.

Dynamic load effects on bridges are characterised by the parameter known as the Impact Factor, I. This is defined as the ratio between the maximum live load effect at a particular point on the bridge during one load event, and the maximum static effect produced by the same load at the same point. In other published work and in many design codes the related parameter Dynamic Load Allowance (DLA) is used, where I = 1.0 + DLA. Figure 1 is reproduced from the Transit New Zealand Bridge Manual (TNZ, 1994) and shows the value of Impact Factor to be used for design of New Zealand road bridges. The criteria are identical to those of "Standard Specifications for Highway Bridges" (AASHTO, 1992). Here, the only parameter governing Impact Factor is the span length. The subject of this project was the Impact Factor for moment in simple and continuous spans, and moment in deck slabs. The question of the values for moment in other members, and for shear and reaction effects, was not addressed.

There have been numerous bridge dynamic load investigations in the past, both in New Zealand and overseas. A comprehensive review of the subject has been presented by Paultre, Chaallal and Proulx (1992), and extensive reference has been made to that paper while writing this report. Some investigations have been theoretical treatments of the subject, and some have involved physical measurements. However, it appears that most of the latter have made use of test vehicles under controlled conditions, and consequently, the results can be said to be somewhat artificial. The major exceptions are the investigations carried out in connection with development of the Ontario Highway Bridge Design Code (OHBDC 1983, 1992), by Wright and Green (1963), Csagoly et al (1972) and Billing (1984).

Previous unpublished work at Works Consultancy Services Central Laboratories indicated that there was a significant relationship between load intensity and Impact Factor. However, this aspect does not seem to have featured prominently in work published elsewhere, except for the general observation that very light vehicles often produce large impact effects, (e.g. Bakht and Pinjarkar, 1989). Logically, an Impact Factor adopted for design purposes should relate specifically to the design load intensity, and therefore, even if values for other load levels are significantly different, they are not relevant so long as they do not produce a total effect greater than the design load with design Impact Factor. The design load specified in the Transit New Zealand Bridge Manual (TNZ 1994) is designated HN (for Highway Normal), and is shown diagrammatically in Figure 2. In the work described below, the intensity of each load event has been related to the HN load, by determining the percentage of HN moment produced on the span in question.

Another feature of reported investigations was that even those which measured the effects of normal traffic had only done so for a limited time. In general it is more appropriate to leave recording equipment in place for an extended period so that a statistically reliable measure of real traffic effects can be gathered as was done for this study.

This procedure recorded the effects of the variables of vehicle speed, position in the lane, multiple presence on the span, and the whole range of vehicle dynamic characteristics. These are all difficult to simulate adequately under test loads or during a limited investigation.

1.2 Objectives

The objectives of this project were:

- To examine the available literature to determine if there was sufficient data which could be confidently applied to New Zealand conditions, and would either verify existing criteria or justify a change to them, and, if there was not, the following three tasks would be performed;
- To acquire instrumentation to investigate the dynamic effects of traffic under normal operating conditions;
- To install the instrumentation on a selection of bridges for sufficient time to obtain a statistically significant quantity of data, including measurement of all parameters considered likely to influence the dynamic response; and
- To analyse the data and either verify the appropriateness of existing design impact criteria, both for main members and for deck slabs, or propose alternative criteria.

1.3 Overall Procedure

A literature search examined available research records, but concluded that there were likely to be significant differences between New Zealand traffic and that of the countries where investigations had taken place, and that as load intensity was not usually defined well enough to be able to correlate results, local testing would be justified.

The next task was to obtain suitable recording equipment. Enquiries were made of other researchers to see if any existing equipment could be used, but the conclusion was that it would be more satisfactory to develop equipment specifically for the task, and this was the course taken. The enquiries made are described in Appendix A.

The equipment was developed and deployed on twelve bridges, and more than 22,000 loading events were logged over a period of about eleven months, between January and November 1993. The spans of the bridges investigated ranged between 9 m and 30 m.

The instrumentation, the bridges investigated and the analysis of results are described below.

2 INSTRUMENTATION

2.1 Survey of Logging Systems Available

A survey of available literature was made to determine whether suitable equipment was available to measure bridge response to a large sample of traffic operating under normal conditions. Discussions were held with the authors of references which described equipment likely to be suitable, but the conclusion was that it would be more convenient and economical to develop such equipment from scratch. Appendix A describes the literature search, discussions held and conclusions reached.

2.2 Development of Data Logger

Following the discussions described in 2.1, the decision was made to base the recording equipment around a laptop computer. Measurements of strain due to bending were recorded, and consisted of the full wave form response of the bridge to each vehicle as it crossed. In this way, there was complete freedom subsequently to analyse the response in any appropriate way. The equipment developed is described in Appendix B.

3 FIELD WORK

3.1 Selection of Bridges

The objective in selecting the bridges to be investigated was that they should cover all the common types of construction, and be representative of the commonest span lengths. Accordingly, with respect to the main supporting members, four were of reinforced concrete, four were of prestressed concrete and four were of structural steel. The span lengths ranged from 9.15m to 30.0m. Two were slab spans, seven were beam and slab construction, one was a truss bridge, one had twin box girders and one consisted of multiple adjacent precast units. With respect to structural form, six were simple spans, four were end spans of continuous bridges, one was an interior span of a continuous bridge, and one an anchor span in a bridge which had pairs of hinges in alternate spans. All bridges were on normal two lane highways with a 100 km/h speed limit.

Other constraints on the choice of bridges were:

- That they should be close enough to the laboratory to avoid more than one overnight stopover when installing and retrieving the equipment;
- That there should be reasonably easy access to the bridge soffit, to avoid the necessity of hiring special equipment for this purpose; and
- That there should be somewhere to install the data logger in a reasonably secure position to avoid vandalism.

Several potential bridges were discarded due to failure to satisfy one or more of these constraints.

Details of the bridges selected, and the parameters considered in this investigation for their influence on the Impact Factor, are contained in Table 1.

3.2 Installation of Recording Equipment

Strainarms with a nominal length of 300 mm were attached, as near as possible to midspan, to each beam, truss or girder soffit, and on slab bridges at intervals across the width of the bridge. Strainarms were also attached to one deck slab at the midpoint of the slab span, to record strains due to transverse bending in the slab. On concrete surfaces, they were attached with concrete anchors drilled in, using dental plaster as a levelling medium on uneven surfaces. On steel surfaces they were attached either by clamps or by adhesive.

The data logger was installed as far as possible out of reach of interference, and was contained in a locked cabinet.

Traffic sensors were installed on the approaches to the span, usually entirely outside the span, in the direction of approaching traffic on both lanes.

3.3 Calibration of Bridge and Equipment

In order to calibrate each installation, a heavy vehicle with known axle loads and spacings was used, and the percentage of design load moment which it produced on the span length was calculated, assuming a simple span. The vehicle chosen for each bridge was as heavy as could be locally obtained, so as to induce as high a percentage of design load as possible in the bridge, and thus reduce the calibration error. The proportion of design load produced by the calibration vehicles, assuming simple spans, varied between 29% and 56%. The vehicle was driven over the bridge a number of times in both lanes at crawl speed, so as to minimise the impact effect as far as possible. At the point of maximum strain during the passage of the vehicle, the strain readings from all the channels recording main member bending strain were added together. The mean of this value for the calibration vehicle in each lane was taken as the calibration value for the bridge. Thus any vehicle which produced a sum of the strain readings equal to the calibration value was taken to produce the same percentage of HN as the calibration vehicle. The sum of the strains from all channels was not sensitive to vehicle lateral position, and this eliminated the effects of eccentricity of loading.

The calibration procedure produced an error for a span which was not simply supported, because the shape of the influence line for moment at midspan was different from that of the simple span. The effect of this was examined by comparing the percentage of HN produced by the calibration truck on the two influence lines, for various continuous beam cases. For a typical end span, the difference was of the order of 3.5%, and for a typical interior span, 5.5%, and corresponding errors exist in the percentage of HN recorded in the data. For the Otaki River Bridge, the only interior span, the error was corrected by modifying all the percentage HN values by the required factor. The error in the end span cases was accepted as it had little effect on the results.

Because of the two way action in the deck slabs, and the considerable influence of the load sharing between beams, it was not possible to obtain consistent slab strains between successive passes of the calibration truck. The attempt to calibrate the deck slab strainarms was therefore abandoned.

3.4 Acquisition of Data

The intention was to leave the equipment installed until approximately 2000 heavy vehicle loading events had been recorded. As it turned out, this was not achieved in some cases for several reasons. Early in the project, considerable difficulty was experienced in attaching securely the traffic sensors to the road surface, because of wet conditions. Failure of the sensors themselves occurred in some cases. At Makohine Stream Bridge, battery failure was experienced, and problems with two strain arms meant that only southbound vehicles were analysed. These problems sometimes led to acceptance of a lesser number of loading events, but this did not affect the viability of the project.

4 ANALYSIS OF RESULTS FROM MAIN BEAMS

4.1 Initial Reduction of Raw Data

4.1.1 Impact Factor for One Beam

The raw data for each vehicle event was in the form of strain variation with time for each channel recorded, from the time the vehicle activated the first traffic sensor, for the duration of the event, together with the speed recorded for each axle. The first task was to determine the precise method of calculating the Impact Factor for one beam from the strain record. The basic requirement was to determine the maximum strain during passage of the vehicle, together with the equivalent static response (mean strain) at the same instant, assuming that the bridge was oscillating at about its natural frequency. This is feasible when the period of oscillation of the bridge is very much shorter than the time the vehicle takes to cross the span. But at normal highway speeds and for spans of the length investigated, there were typically only one or two cycles of oscillation of the bridge which were anywhere near the peak of the equivalent static response during the passage of the vehicle. This meant that it was difficult to plot the mean strain on the graph manually, and even more difficult to develop an algorithm to perform the task.

For example, if the span was 20m long and had a natural frequency of 5Hz, and it was crossed by a vehicle travelling at 100km/h, the wavelength of the bridge oscillations superimposed on the equivalent static response curve would be 5.6m, or 28% of the span length. This meant that not more than two cycles would be of use to determine the static response.

The solution adopted was to identify the two highest peaks on the strain record, and the lowest trough between them, as illustrated in Figure 3.

Then:

Mean of two highest peaks	= **a**
Mean of **a** and lowest trough	= **b**
Impact Factor	= **a/b**

The record was only accepted if the lower peak was greater than one third the height of the upper peak, and the time between the peaks was less than 1.5 times the natural period of the bridge.

A representative selection of records was examined and the Impact Factor determined both by hand and by the above method, and the difference was not more than 10%, which was considered acceptable.

The alternative of digital filtering of the record to remove the bridge oscillations and obtain the static response, as recommended by Billing and Agarwal (1990), was considered to be less reliable on the short spans investigated.

4.1.2 Effective Impact Factor for the Bridge

The Impact Factors for the different beams of a bridge for one vehicle event are markedly different, so a method of determining the representative value for a bridge, which would be appropriate for design use and could be considered for statistical purposes, had to be chosen. Figures 4 and 5 show the Impact Factors determined as in 4.1.1 above, produced by one loading event on each beam of two bridges. Also plotted are the corresponding maximum beam strains for each beam, and the Impact Factor calculated from the sum of all beam strains.

The plots show that the individual beam Impact Factors for one event vary over a wide range. The value associated with the maximum beam strain is the significant one, and one option would be to use this as the effective value for the event. However, this value is in all cases very close to the value calculated using total strain for all beams. It was considered that use of the value associated with total strain would be more significant as a parameter for the bridge as a whole, and this was the value adopted for each event.

4.2 Processing of Raw Data

The data for each bridge was processed into a file consisting of one line for each loading event. Individual vehicles were identified by matching axle speeds, but where two vehicles travelled closely at the same speed, it was not always possible to separate them. The file contained the following information:

- Truck identification;
- Lane identification;
- Number of axles;
- Speed;
- Impact Factor;
- Percentage of design loading, HN;
- Factor indicating the asymmetry between the two strain peaks used to determine I;
- Time between peaks;
- Maximum axle speed of event;
- Minimum axle speed of event; and
- Percentage of axles matched for speed.

In some cases the processing did not produce a sensible result. Specifically, when the program could not identify the peaks and trough, described in 4.1.1 above, the Impact Factor was recorded as zero. The method of distinguishing between successive vehicles was to examine the speed of successive axles, and when a significant difference was detected, a new vehicle was assumed. However, where a convoy of vehicles closely followed each other at the same speed, it was not possible to identify where one vehicle ended and the next one started. Such an event was recorded as one vehicle with sometimes a very large number of axles. This would only affect the statistics to the extent that some vehicles would be omitted from the record, in favour of the vehicle which produced the largest response in the convoy.

4.3 Parameters Considered in the Analysis

The bridge parameters considered were as follows:

- Span length;
- Span natural frequency;
- Span critical damping ratio;
- Material of main supporting members;
- Span end conditions; and
- Roughness of bridge approaches.

For each bridge, the Natural Frequency and the percentage of Critical Damping were determined where possible from the plot of an individual vehicle record, after the vehicle had left the span. In some cases assessment of the damping was not possible, as the vibration decay was not apparent clearly enough in the records.

Vehicle parameters considered were:

- Intensity of loading (percentage of design load, HN); and
- Vehicle speed.

4.4 Individual Bridge Data Analysis

A plot of Impact Factor against Percentage HN for each vehicle, was produced for each bridge. These are presented in Figures 6 to 17. They show clearly that the general tendency was for the Impact Factor to decrease with increasing load level. To confirm and quantify this, the 95 percentile value of Impact Factor was calculated for each ten percent band of Percentage HN above 30%. The results of this are also shown superimposed on Figures 6 to 17, and are recorded in Table 2. Finally, the values of 95 percentile Percentage HN times Impact Factor for each ten percent band were plotted against Percentage HN at the centre of each band.

The 95 percentile values referred to above are those which exceed 95% of the values for the events in the load band concerned. This criterion was chosen as it is used in the limit state design context to determine design load levels. The calculation method, which assumed a normal distribution, was as follows:

$$95\%\text{ile value} = \bar{x} + \sigma k$$

where $\bar{x}$ is the mean value for events in the load band
σ is the standard deviation for the events
k is a one sided tolerance limit factor for 95% confidence, dependent on the number of events in the load band.

4.5 Correlation of Individual Bridge Data Analyses

4.5.1 Impact Factor at a Fixed Load Intensity

(a) From the plots described in 4.4, the 95 percentile Impact Factor can be read off for each bridge, at any consistent load level. The highest level for which there is a value for nearly every bridge is 85% HN, and these values have been plotted against bridge natural frequency in Figures 18 to 20. For Waikanae River Bridge, an extrapolated value has been used. The values are also presented in Table 2, together with the number of loading events which contributed to each. Also noted on Figures 18 to 20 against each plotted point is one of the parameters listed below:

- In Figure 18, critical damping ratio
- In Figure 19, material of main supporting members
- In Figure 20, span end conditions

For comparison with current design criteria, the values of Impact Factor laid down in the Bridge Manual (TNZ 1994) have also been plotted. As the Bridge Manual values are related only to span length, L, the following nominal relationship between this and natural frequency, f_o, has been assumed for the purpose of the plot:

$$f_o = 82\ L^{-0.9}$$

This is taken from Paultre et al (1992), and is based on tests on more than 200 European bridges. Equivalent values from the Ontario Highway Bridge Design code (OHBDC, 1983) have also been shown.

(b) In order to test the correlation of Impact Factor with road roughness in each lane, the results for loads in the 85%HN load band for each bridge were split up between the two lanes, to give a 95 percentile Impact Factor for each lane. In Figure 21, these values have been plotted against natural frequency, with the lane roughness in NAASRA counts per kilometre noted beside each point. NAASRA counts are the number of units of 15.2 mm relative vertical movement between the axle and the chassis, recorded by a standard vehicle, travelling at a standard speed, per kilometre of lane. A value is calculated every 100 m, and appears on the record at the end of the 100 m length.

This does not identify the position of a sudden irregularity, such as frequently appears at a bridge abutment, since its effect is averaged out over the 100 m interval, but it does give a general indication of surface condition.

(c) In order to test the correlation of Impact Factor with span length, the 95 percentile values for loads in the 85%HN band were plotted against span in Figure 22 together with the Bridge Manual criteria (TNZ, 1994).

(d) In order to test the correlation of Impact Factor with vehicle speed, the value recorded by each vehicle in the 85%HN load band, for all bridges, has been plotted in Figure 23 against speed. The 95 percentile Impact Factor values have been calculated for each band of 10km/h, and plotted at the centre of the band. In the 65%HN band, one very high value was omitted because it greatly distorted the result.

4.5.2 Comparison of Impact Factor at All Load Intensities

In order to compare the Impact Factor values from all bridges over the whole range of loads, the 95 percentile values from each 10% band of HN loading were plotted against percentage of HN in Figure 24. No attempt was made to correlate these values with any of the bridge or traffic parameters used previously. For comparison with current design criteria, the range of design Impact Factor laid down in the Bridge Manual (TNZ, 1994) is shown at 100%HN.

5 DISCUSSION AND CONCLUSIONS FROM MAIN BEAM RESULTS

5.1 Individual Bridge Analyses

The first conclusion from the initial individual bridge analyses was that on 10 out of the 12 bridges, the critical dynamic load (as indicated by the 95 percentile value of Percentage HN times Impact Factor) was generated from the highest static load. This might be thought to be obvious, but there was a possibility that some lesser load might have a large enough Impact Factor to become more critical than a higher load with a small Impact Factor. For Makohine Stream and Waikanae River Bridges, this was so for the loads recorded, but on these bridges the maximum loads were well under the 100%HN level, and it was probable that if larger loads were recorded, they would generate the critical dynamic total.

In spite of the large number of events recorded, the actual number in the region of greatest interest, that is around 100% HN load level, for each bridge was relatively small. The statistical confidence in the result recorded for each 10% band was 95% as stated, but since the calculation involved a factor dependent on the sample number, it was likely that if the number of events recorded had been larger, the 95 percentile values of Impact Factor at this load level would be lower. The recorded results were therefore likely to be conservative. In any future work, use of a trigger level of say 50%HN would allow equipment to remain in place for a longer period for the same power requirements, and thus record more vehicles at the heavy end of the range.

5.2 Correlation Between Bridges at Fixed Load Intensity

A study of Figures 18 to 22 shows that there is generally poor correlation between the Impact Factors for the various bridges on the criteria shown.

The strongest correlation is with material, in Figure 19, where results from bridges of reinforced concrete, prestressed concrete and structural steel are grouped together in three bands. This is contrary to findings elsewhere, (Paultre et al, 1992).

There is also some correlation with span end conditions in Figure 20, where the simple spans and continuous end spans form one group, and continuous interior spans form another group.

In Figure 21, there appears to be no correlation with the road roughness parameter used, which was the only data available at the time. This parameter is the sum of test vehicle vertical displacements over a standard length of road. However, since other researchers concluded that road roughness was a significant parameter, (Paultre et al, 1992), if any further work is done on this subject, a study of the method of measuring road roughness for this purpose should be made to see if it can be better quantified. It may be sufficient to use the same method, but the sum of the test vehicle displacements could be taken over a shorter length of the bridge approach than the standard 100m. This would mean that the effect of a major irregularity such as that due to an expansion joint, or to approach fill settlement, would have a much greater influence on the resulting parameter.

In Figure 18, there is no correlation with the damping factors measured, but it is highly likely that damping at high load levels is considerably different from that at no load, which is how the values were derived. There appears to be no easy way of determining damping at high load levels, but again, if any future measurements are made, a method could be developed.

In Figure 22, there is no correlation with span.

In Figure 23, there is some indication that Impact Factor increases with speed at the high end of the range.

It should be noted that although the design criteria have been shown on the charts, they do not apply to the 85%HN load intensity, but to 100%HN. Thus the plotted Impact Factor values which are above design value do not in fact relate to critical loads.

5.3 Correlation Between Bridges at All Load Intensities

The 95 percentile Impact Factor values for each 10% range of HN loading, for all bridges, have been plotted in Figure 24. This shows that although there is a wide scatter of values over the lower percentages of HN loading, the variation at 100%HN is comparatively small. The envelope drawn indicates that without taking any account of bridge or traffic parameters, the Impact Factor at 100%HN is likely to be less than 1.2. This conclusion is in line with the recently published "AASHTO LRFD Bridge Design Specifications" (AASHTO, 1994). This specified a value of 1.33 for all bridges (for main members).

The commentary stated that the actual likely maximum was 1.25, but the fractional part of the parameter was multiplied by a factor of 4/3 for design purposes.

The conclusion is also in line with the "Ontario Highway Bridge Design Code" OHBDC, 1992), which specified values dependent only on the number of axles of the design truck causing the load, and not on any bridge parameters.

5.4 Conclusions from Main Member Results

The conclusions from this study are:

(a) Although there was little correlation of Impact Factor with any of the parameters plotted, the results did indicate that the current criteria were conservative at design load level. It is unlikely that further work would indicate changes which would have a significant economic effect.

(b) The Impact Factor for lower load intensities was highly bridge dependent, although the criteria were not identified.

(c) The equipment developed for this study would be of great value in investigating the impact induced in a critical bridge for the purposes of evaluating actual load capacity, whether at normal load or at overload intensity. It would enable a realistic value to be used with confidence, knowing that it would be specific to the bridge.

6 DISCUSSION AND CONCLUSIONS FROM DECK SLAB RESULTS

6.1 Procedures Followed

The Impact Factor for each slab loading event was derived from the strain record of the strainarm attached to the slab, in the same way as described for beams in 4.1.1. The results are plotted in Figures 25 to 32 for each bridge having a deck slab. All such bridges in this investigation had strain records due to positive (sagging) moments in the slab. Three of them also had a significant number of records due to negative (hogging) moments. Positive moments are caused by loads directly on the slab, whereas negative moments are caused by loads on some other part of the bridge. Both sets of records are plotted on the same figure in each case, and for consistency the plots for those bridges without negative moment records are drawn to the same layout.

As explained in 3.3, it was not possible to calibrate the deck slab strainarms by the method intended. The Impact for each event has therefore been plotted against the strain parameter equivalent to the maximum static load effect for that event, defined as b in Figure 3. These strain parameters are scattered over a recognisable range for each bridge, (ignoring some large outlying values which were assumed to be erroneous). The maximum value for each bridge was therefore assumed to represent the strain caused by the heaviest axle or axle group in the vehicle population. The scale of each plot has been adjusted so that the maximum value for positive moments is approximately equal for each bridge, regardless of its numerical value. The numerical values of the strain parameter are dependent on the stiffness of the slab.

Also shown on each figure is a sketch of the bridge cross-section, showing the position of the strainarm.

The 95 percentile values for Impact Factor have been calculated for the events with strain parameters in the top 95% of the range for each bridge, divided into four equal bands. In most cases the bottom 5% of the range includes some very large Impact Factors, which are not considered to be relevant, and are possibly erroneous because it is likely the algorithm described in Figure 3 does not work well on small strains. The 95 percentile values are plotted at the centre of each band.

As can be seen, the 95 percentile values of slab Impact Factor show a marked tendency to decrease with increasing load, in the same way as the values for main beams. In order to compare all the bridges, these values have been plotted together in Figure 33.

6.2 Discussion and Conclusions from Deck Slab Results

Figure 33 shows that all the bridge slabs behave in a similar manner in response to dynamic loads. The 95 percentile values of Impact Factor for the maximum load band only exceed the design value of 1.30 (TNZ, 1994) in one case, that is Paekakriki Overbridge, where it is 1.35. It is likely that the maximum load band represents axle loads of around 80kN, so extrapolating to the HN design axle load of 120kN indicates that 1.30 is still an appropriate design value for interior slabs. In order to verify that it is also appropriate for cantilever slabs, further work would be required.

The existence of significant negative moments was unexpected. Even more unexpected was that in one case (Paekakariki Overbridge) the magnitude of the negative moment range was approximately equal to that of the positive moments. It is evident from consideration of the records and of the bridge details that:

(a) Positive moments are caused by loads directly on the slab, whereas negative moments are caused generally by loads in the adjacent lane.

(b) None of the bridges on which significant negative moments were recorded have intermediate diaphragms or cross-frames. In only one case of a bridge with intermediate diaphragms (Porewa Stream Bridge at Rata) were any negative moments recorded, and these were insignificant.

It is concluded that intermediate diaphragms effectively prevent transmission of transverse negative moments from one side of the bridge to the other by stopping the beams rotating relative to each other.

As can be seen from Figure 33, the Impact Factor characteristics for negative moments were similar to those for positive moments.

7 RECOMMENDATIONS

The following actions are recommended:

(a) That no change should be made at this time to the Impact Factors specified in the Bridge Manual.

(b) That active consideration be given to use of the equipment developed to determine the actual 95 percentile Impact Factors experienced by critical posted bridges and those frequently restrictive for overweight loads.

(c) That if any future work is undertaken, it should include:

   (i) A study of the method of quantifying road roughness, to identify a more appropriate parameter.
   (ii) Use of a trigger level of say 50%HN, in order to increase the useful data for the same power requirements.
   (iii) Development of a more reliable method of identifying separate vehicles.
   (iv) Development of a method of determining critical damping ratio at actual loading levels.
   (v) Instrumentation of some cantilever slabs.

Table 1: Details of Bridges

Name and Route Position	Description	Span Length & Type	Natural Frequency	Critical Damping Factor	NAASRA Road Roughness
Paekakariki Rail Overbridge SH1N 953/0.00	4 in situ RC beams In situ RC Slab	12.2m Simple span	9.0Hz	3%	S 132 N 76
Waikawa River Bridge SH 1N 903/6.53	6 in situ RC beams In situ RC slab	9.15m Simple span	6.0Hz	*	S 103 N 142
Waikanae River Bridge SH1N 931/5.16	In situ PSC slab	18.3m Continuous End span	2.6Hz	*	S 104 N 142
Moonshine Bridge SH2 946/8.84	7 PSC U-beams In situ RC slab	24.0m Simple span	4.6Hz	1%	S 92 N 103
Pakuratahi River Bridge SH2 931/6.81	4 PSC I-beams In situ RC slab	21.7m Simple span	6.7Hz	7%	S 94 N 75
Rimutaka No.1 Bridge SH2 921/1.24	5 steel trusses Precast RC slabs	24.4m Simple span	6.8Hz	2%	S 123 N 83
Otaki River Bridge SH1N 915/6.71	4 in situ RC beams Insitu RC slab	14.3m Continuous Interior span	6.8Hz	*	S 75 N 74
Rangitiki River Bridge (Bulls) SH1N 844/1.27	5 steel beams In situ RC slab	26.8m Anchor span for cantilevers + susp spans	3.4Hz	4%	S 45 N 54
Porewa Stream Br (Rata Factory) SH1N 801/15.80	4 steel beams In situ RC slab	12.8m Continuous End span	2.84Hz	*	S 75 N 85
Porewa Stream Bridge (Ross's) SH1N 801/8.51	9 precast PSC d/c units No deck slab	11.75m simple span	3.0Hz	*	S 94 N 85
Makohine stream Bridge SH1N 780/12.49	In situ RC slab	11.05m Continuous End span	2.5Hz	*	S 83 N 85
Mangatewai-nui River Bridge SH2 758/0.40	2 steel box girders In situ RC slab	30.0m Continuous End span	3.6Hz	3%	S 170 N 170

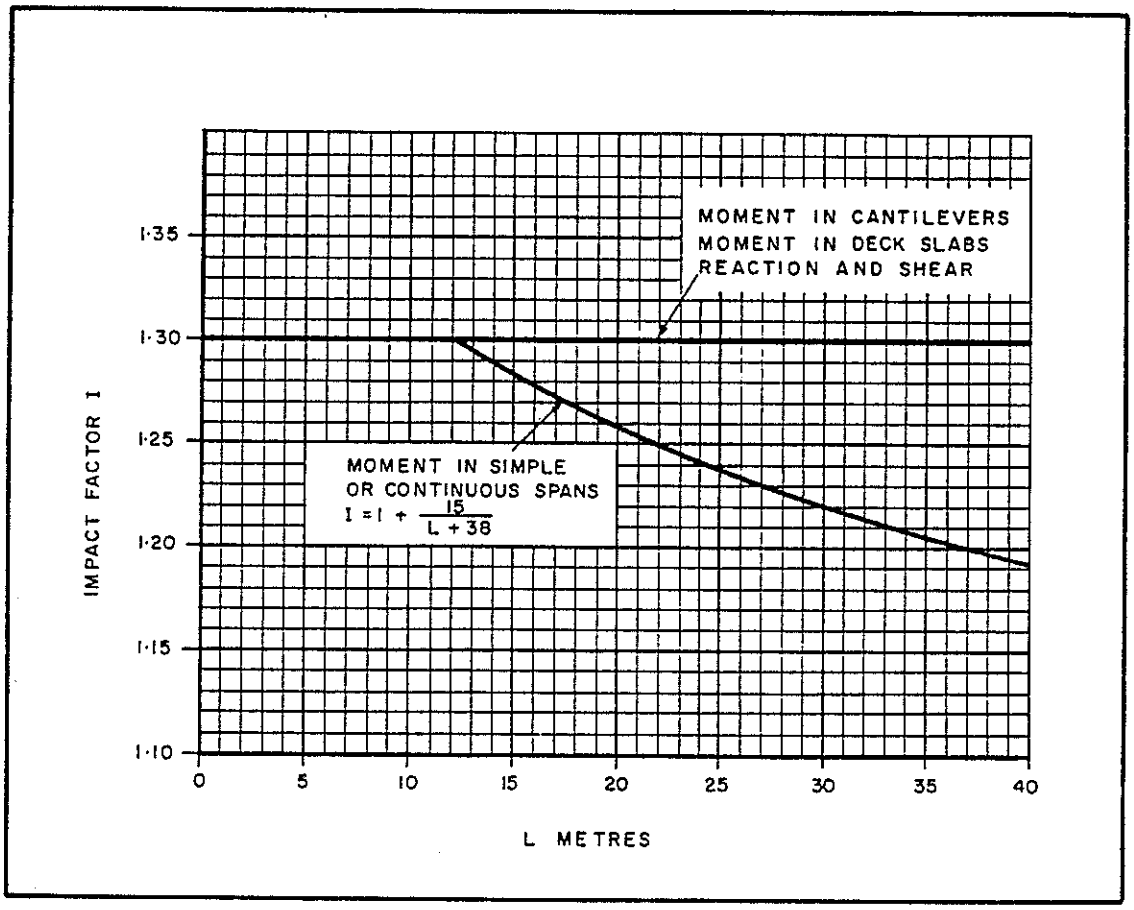
* Value not obtainable S = Southbound N = Northbound

Table 2: Summary of analysis Results

Bridge Name	95%ile Impact Factor, All events									Events 80 to 90%HN					
	Centre of load band, %HN									Number of events			95percentile Impact		
	35	45	55	65	75	85	95	105	115	All traffic	S bound	N bound	All traffic	S bound	N bound
Paekakariki	1.36	1.32	1.27	1.25	1.19	1.12	1.14	1.07		30	12	18	1.12	1.13	1.13
Waikawa	1.86	1.84	1.66	1.58	1.42	1.11	1.08			26	8	18	1.11	1.12	1.12
Waikanae	1.96	1.84	1.77	1.73	1.36					2			*1.20		
Moonshine	1.46	1.37	1.36	1.30	1.25	1.19				23	23	0	1.19	1.19	
Pakuratahi	1.67	1.55	1.41	1.43	1.38	1.37				18	14	4	1.37	1.39	**1.32
Rimutaka	1.50	1.32	1.34	1.24	1.18	1.12	1.06			31	16	15	1.12	1.16	1.08
Otaki	1.38	1.26	1.23	1.16	1.08	1.04				16	7	9	1.04	1.05	1.04
Bulls	1.35	1.33	1.29	1.15	1.05	1.04				19	10	9	1.04	1.04	1.05
Rata	1.64	1.60	1.51	1.37	1.18	1.04	1.02			43	30	13	1.04	1.05	1.03
Rosses	2.03	1.80	1.69	1.58	1.45	1.26				15	12	3	1.26	1.27	**1.52
Makohine	1.64	1.47	1.26	1.15						0	0	0			
Mangatewai-nui	1.34	1.21	1.20	1.12	1.12	1.13	1.04	1.02	1.03	294	205	89	1.13	1.15	1.06

* Extrapolated
**Distorted result due to small sample

Figure 1: Impact Factor for Steel and Concrete Components Above Ground Level and for Bearings



L is the span length for positive moment, and the average of adjacent span lengths for negative moment.

Figure 2: HN-HO-72 Traffic Loading

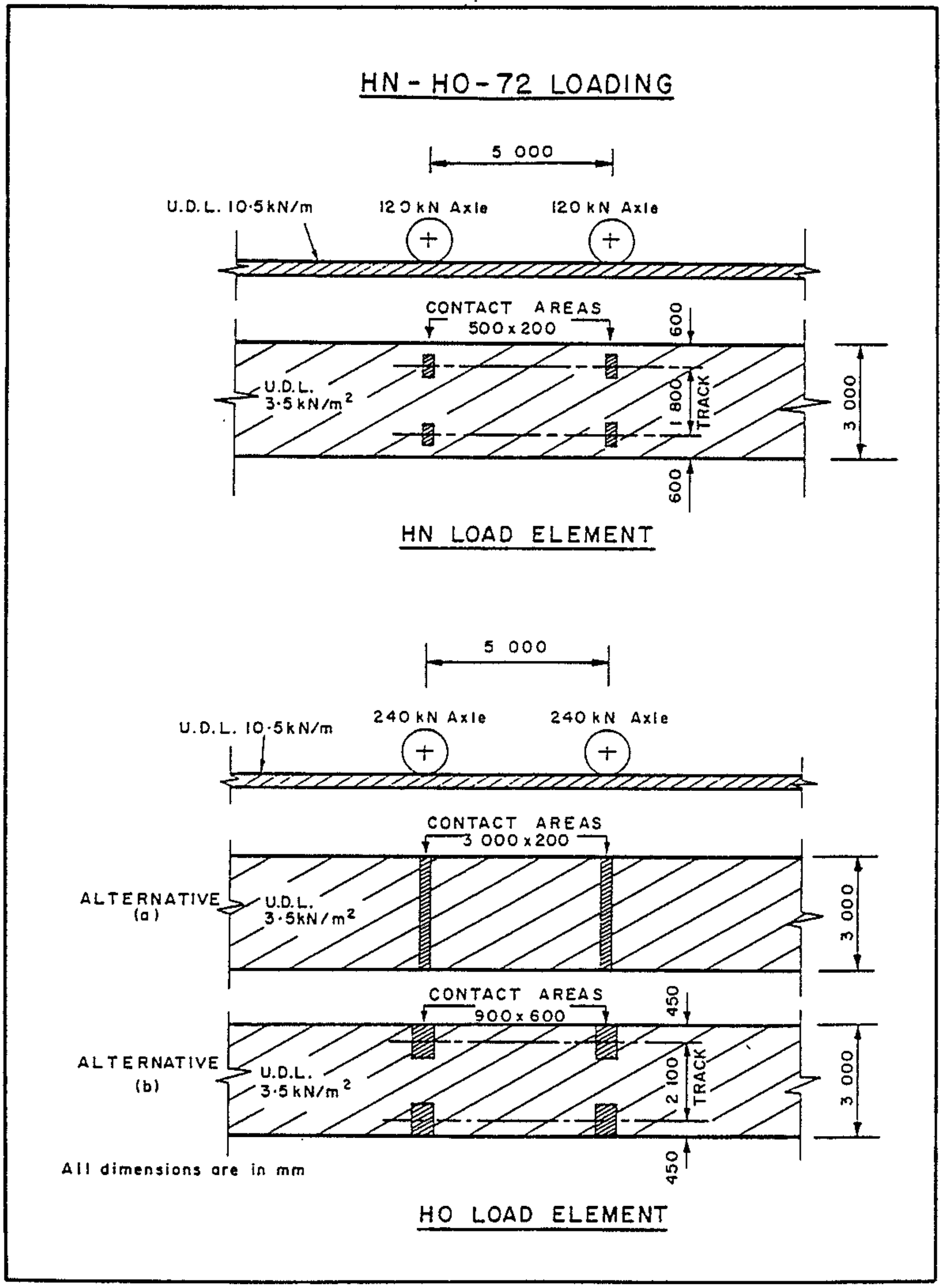
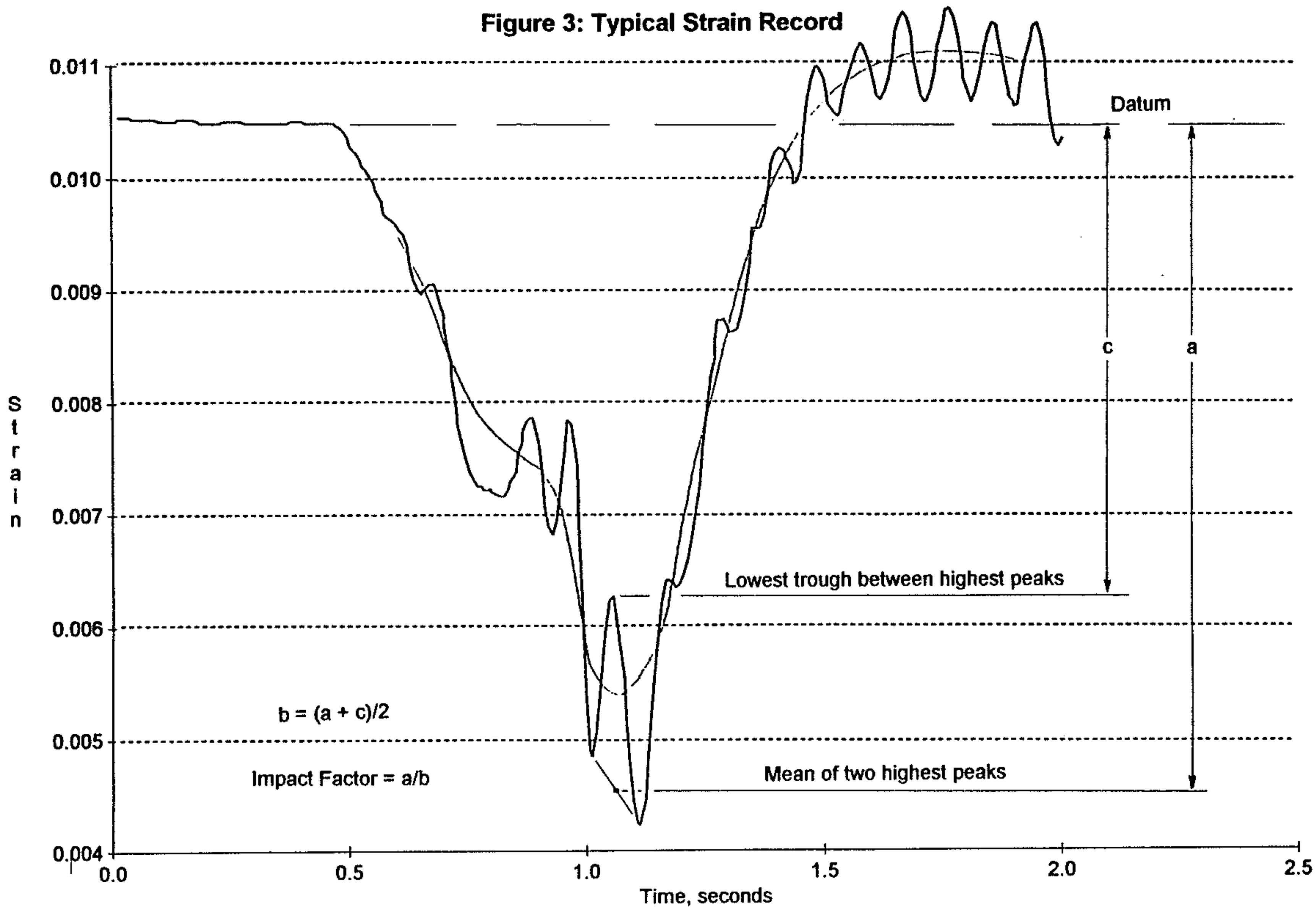


Figure 3: Typical Strain Record



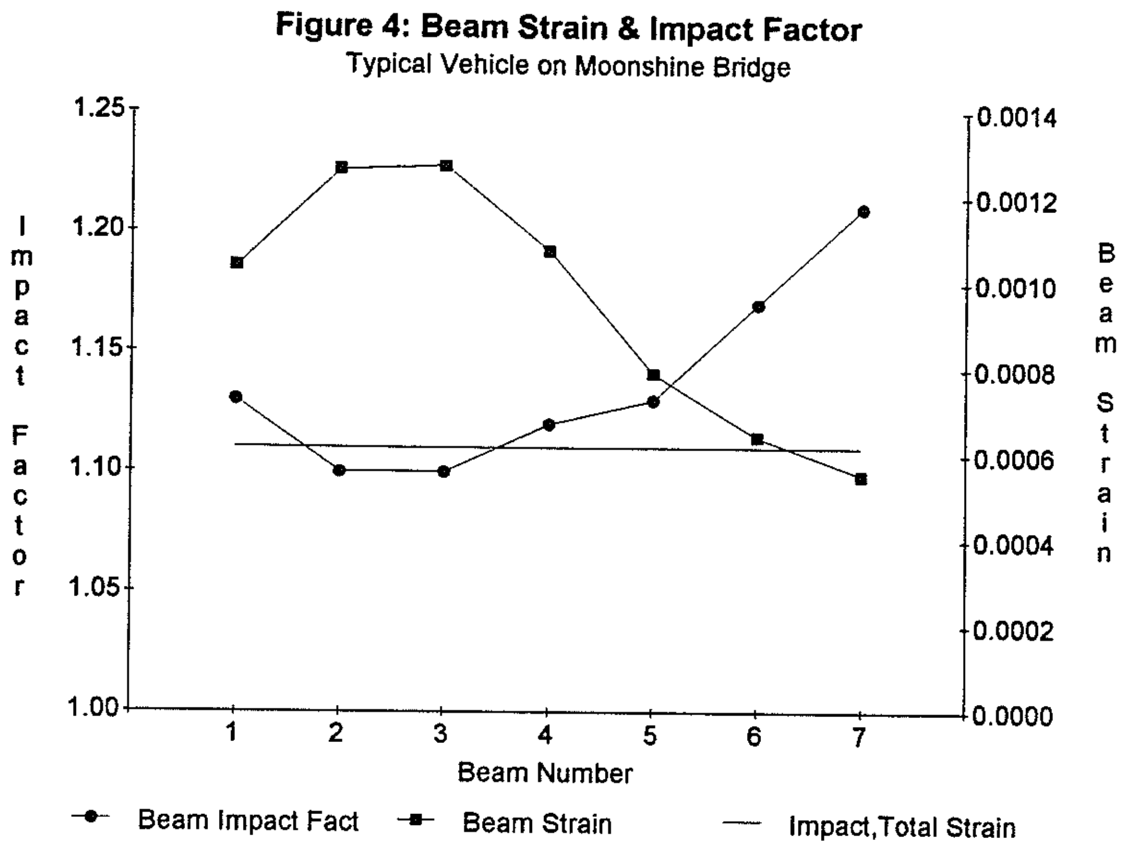


Figure 4: Beam Strain & Impact Factor
Typical Vehicle on Moonshine Bridge
Impact Factor
1.25
1.20
1.15
1.10
1.05
1.00
0.0014
0.0012
0.0010
0.0008
0.0006
0.0004
0.0002
0.0000
Beam Strain
1
2
3
4
5
6
7
Beam Number
Beam Impact Fact
Beam Strain
Impact,Total Strain

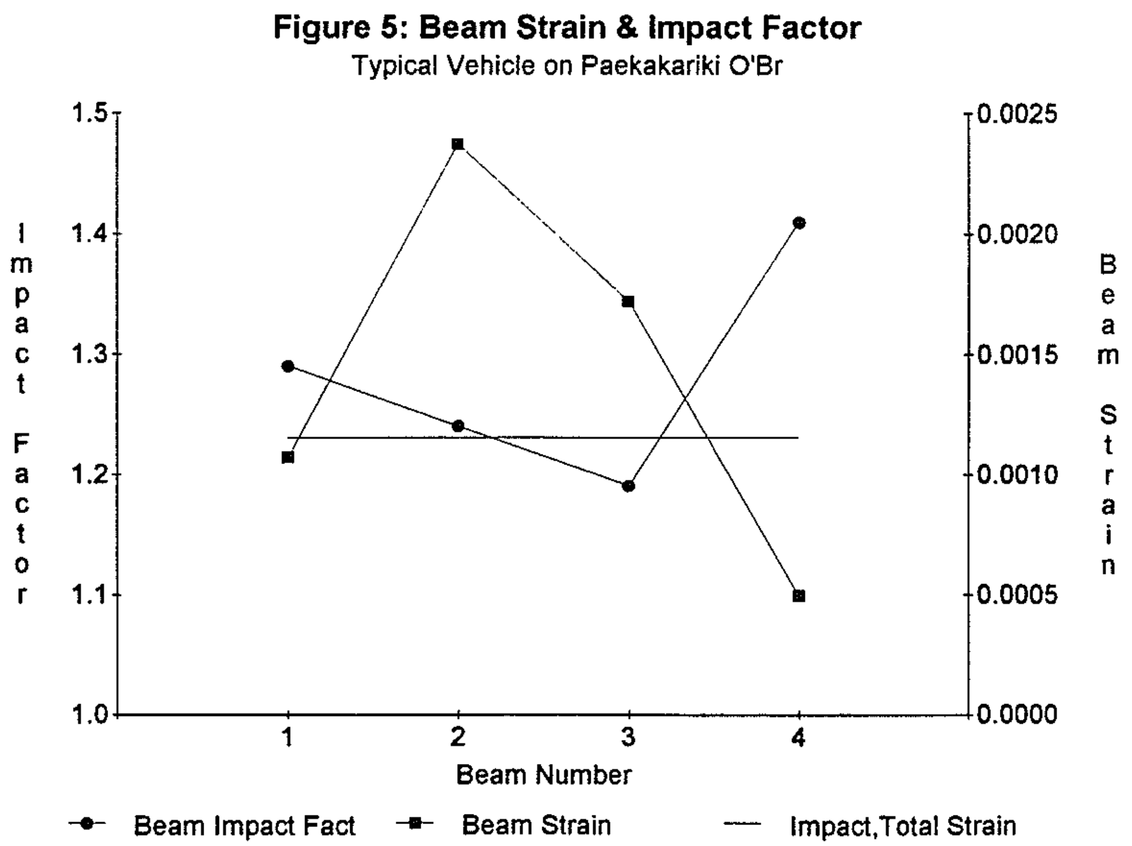


Figure 5: Beam Strain & Impact Factor
Typical Vehicle on Paekakariki O'Br
Impact Factor
1.5
1.4
1.3
1.2
1.1
1.0
0.0025
0.0020
0.0015
0.0010
0.0005
0.0000
Beam Strain
1
2
3
4
Beam Number
Beam Impact Fact
Beam Strain
Impact,Total Strain

Figure 6: Paekakariki Rail Overbridge

Plot of all loading events

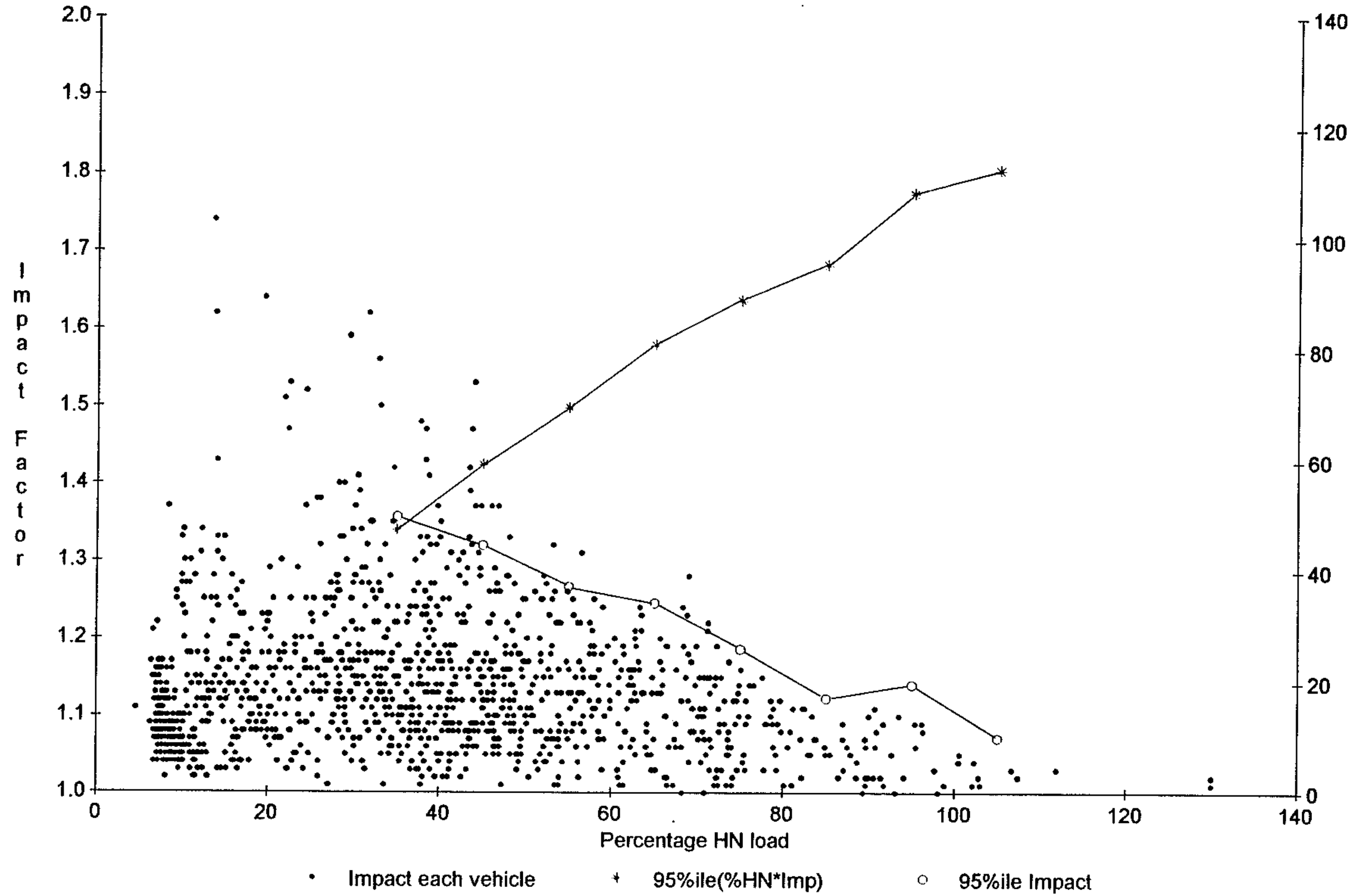


Figure 7: Waikawa River Bridge

Plot of all loading events

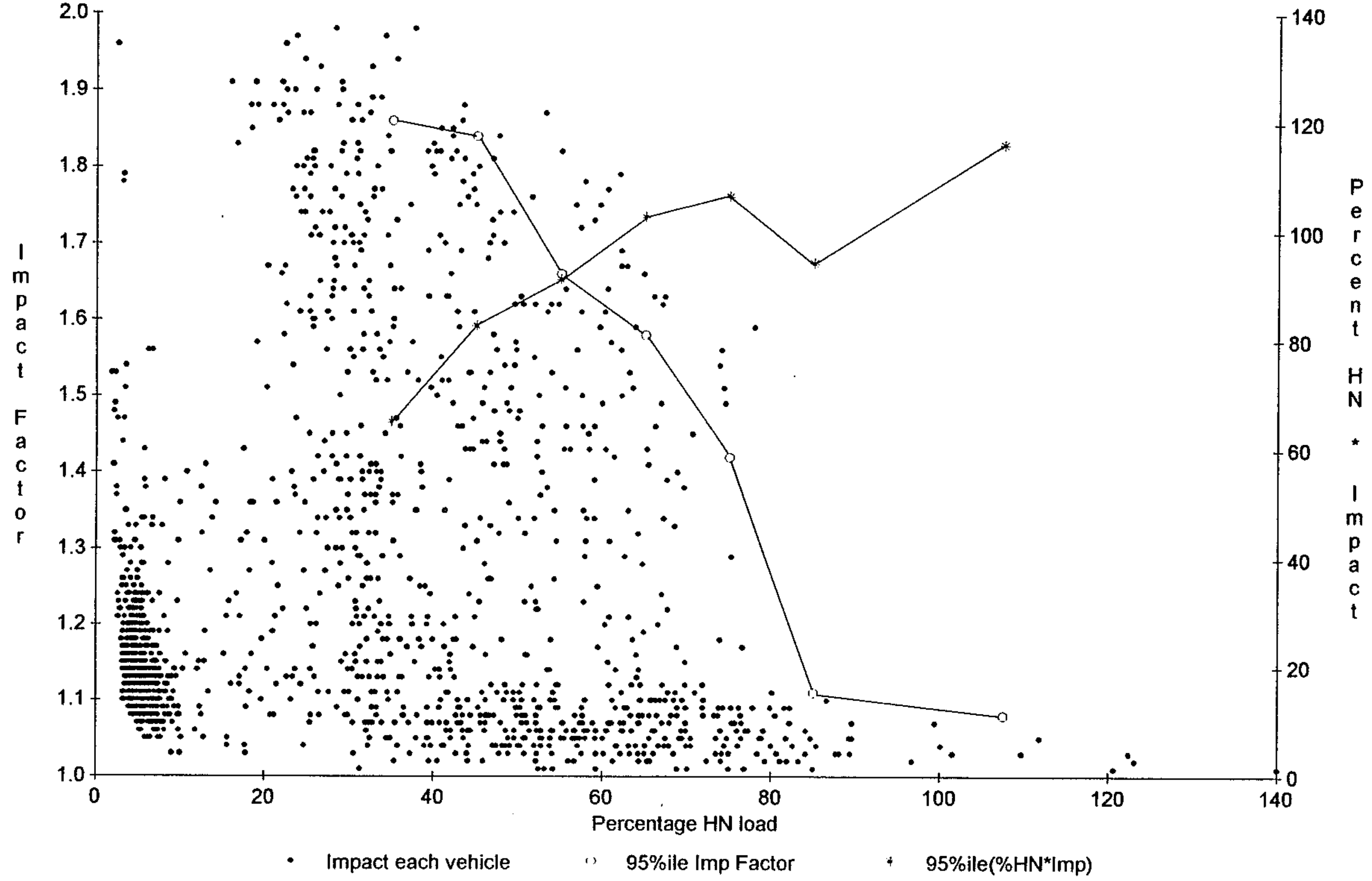


Figure 8: Waikanae River Bridge

Plot of all loading events

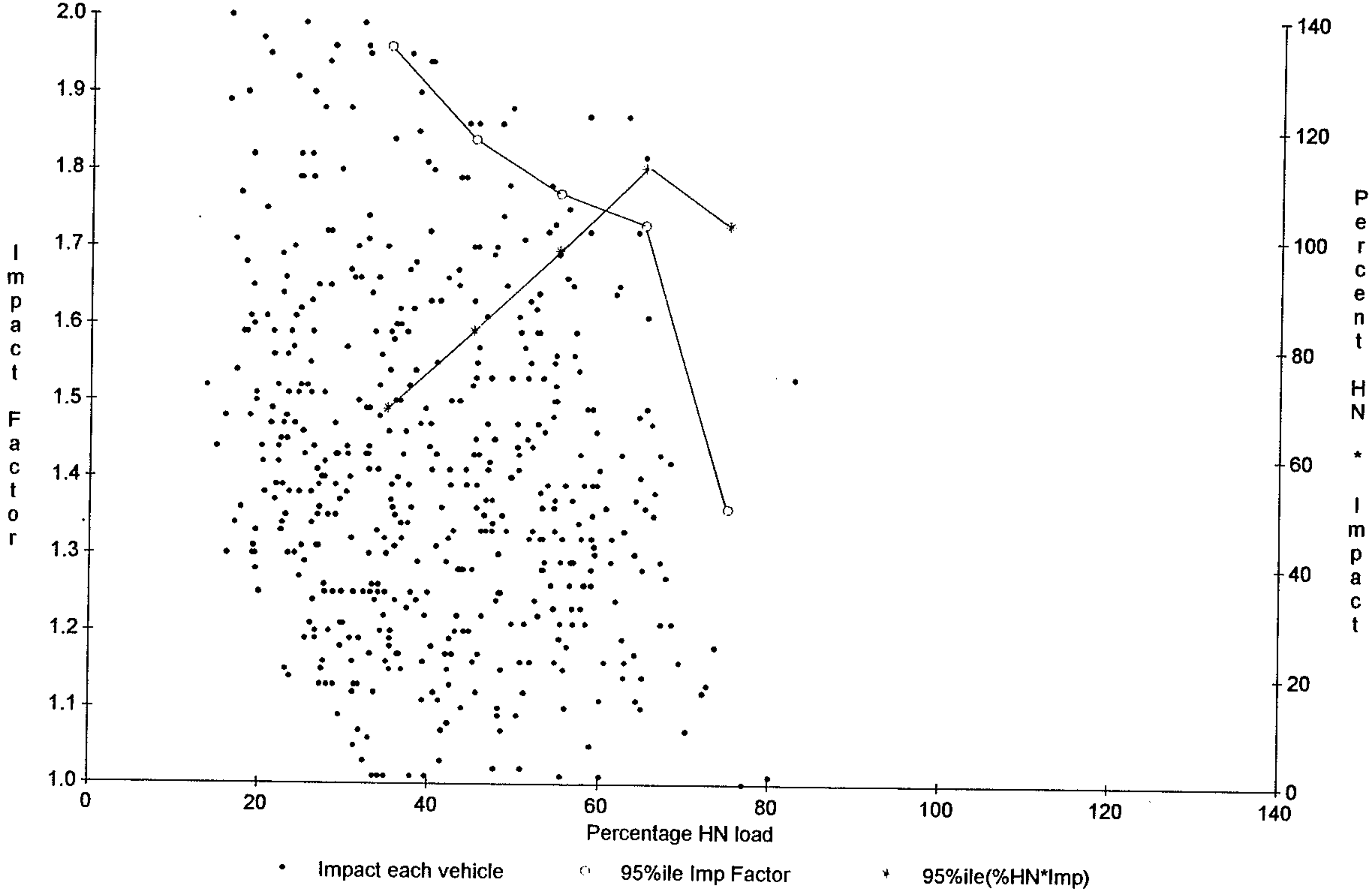


Figure 9: Moonshine Bridge

Plot of all loading events

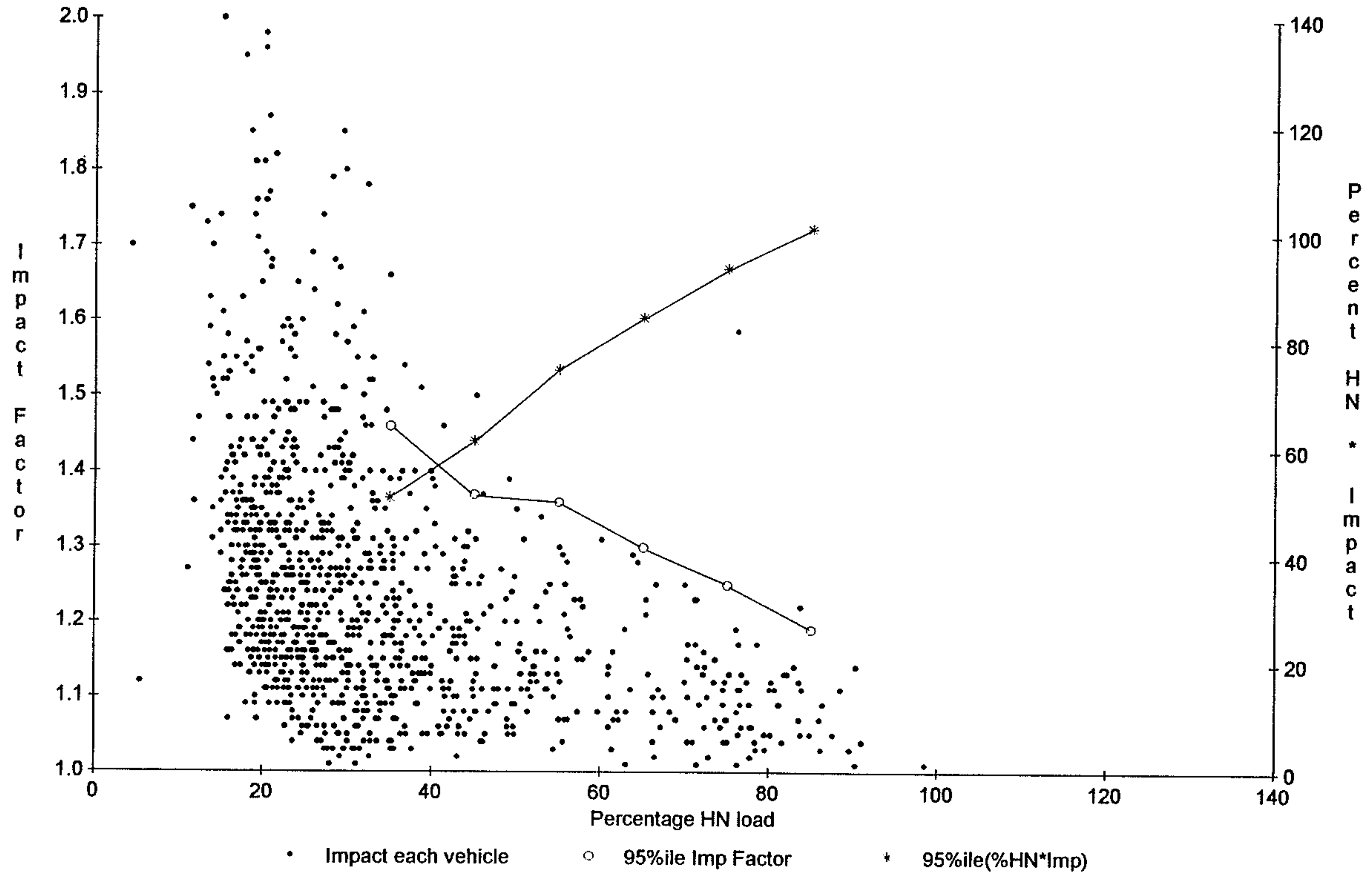
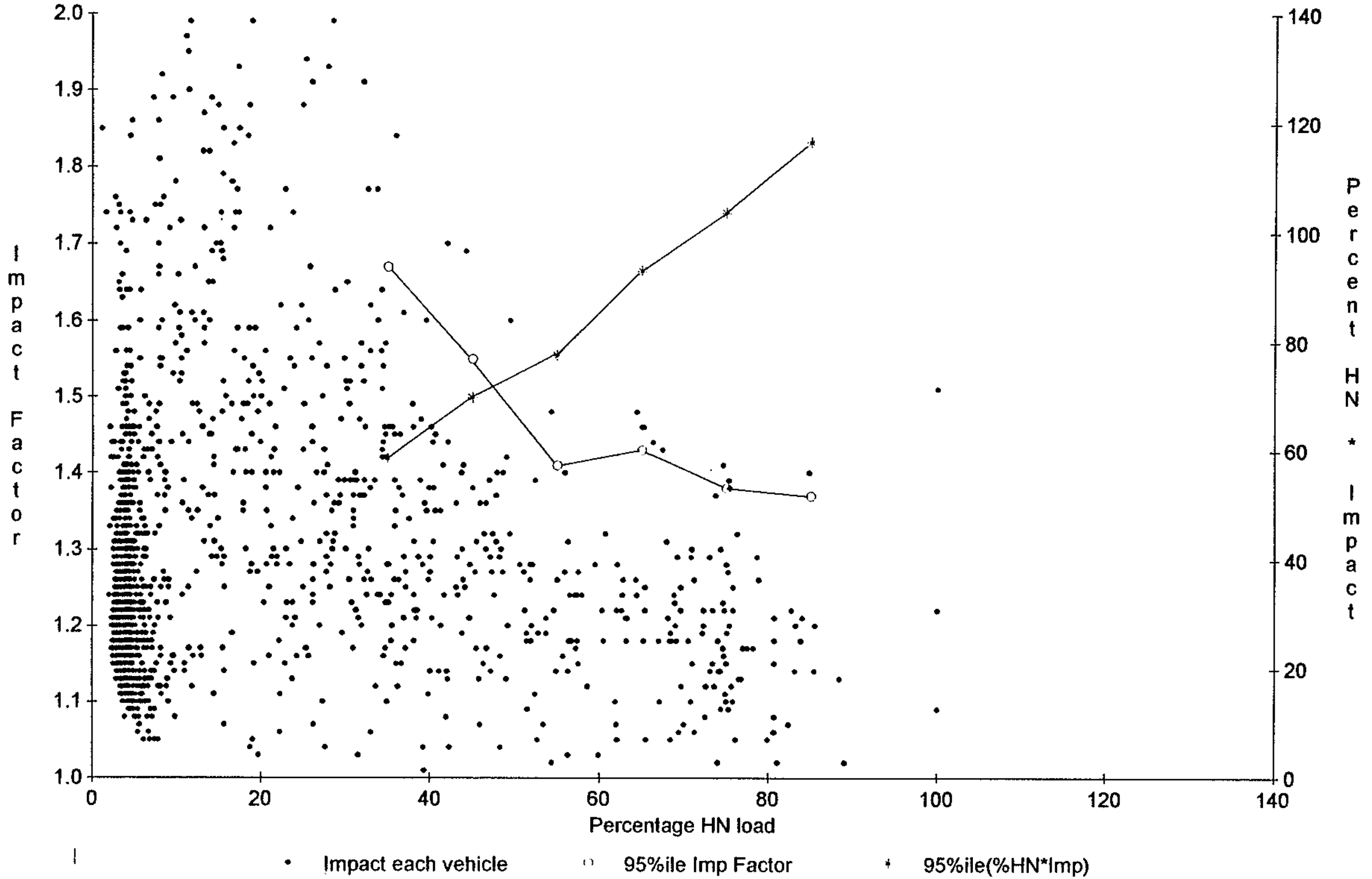


Figure 10: Pakuratahi River Bridge

Plot of all loading events



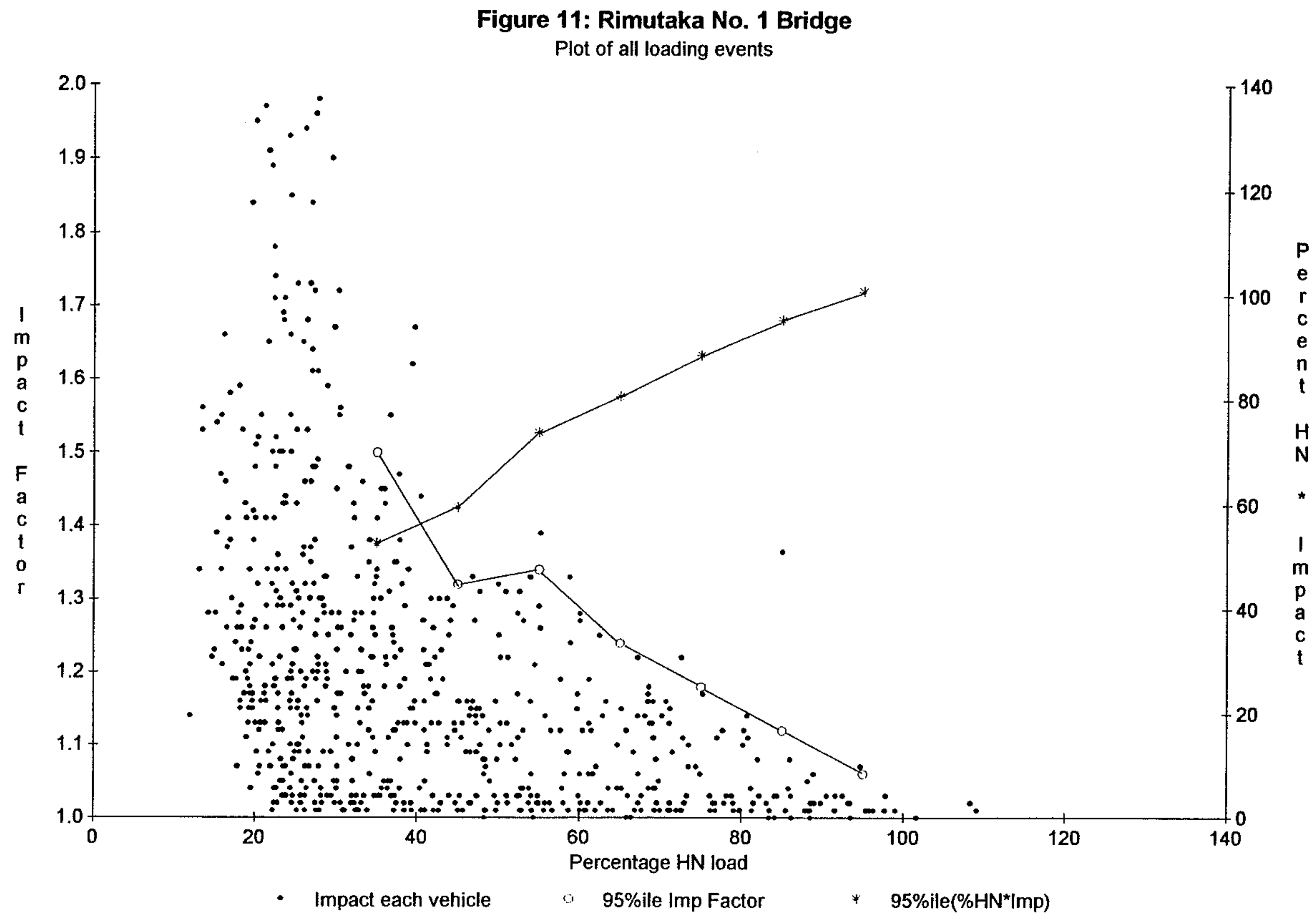


Figure 11: Rimutaka No. 1 Bridge
Plot of all loading events
Impact Factor
Percent HN * Impact
Percentage HN load
Impact each vehicle
95%ile Imp Factor
95%ile(%HN*Imp)
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0
20
40
60
80
100
120
140

Figure 12: Otaki River Bridge

Plot of all loading events

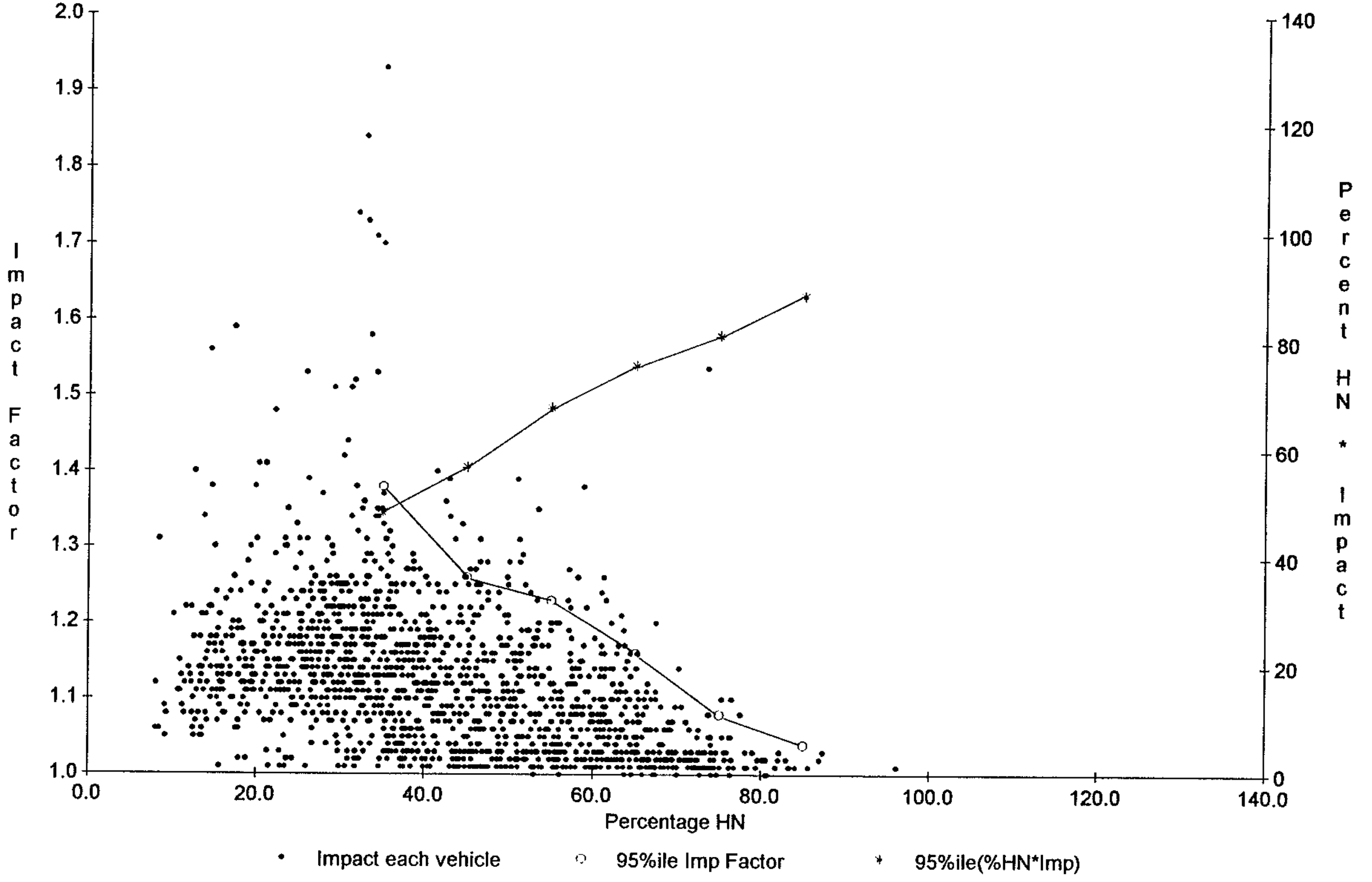


Figure 13: Rangitiki River Br. (Bulls)

Plot of all loading events

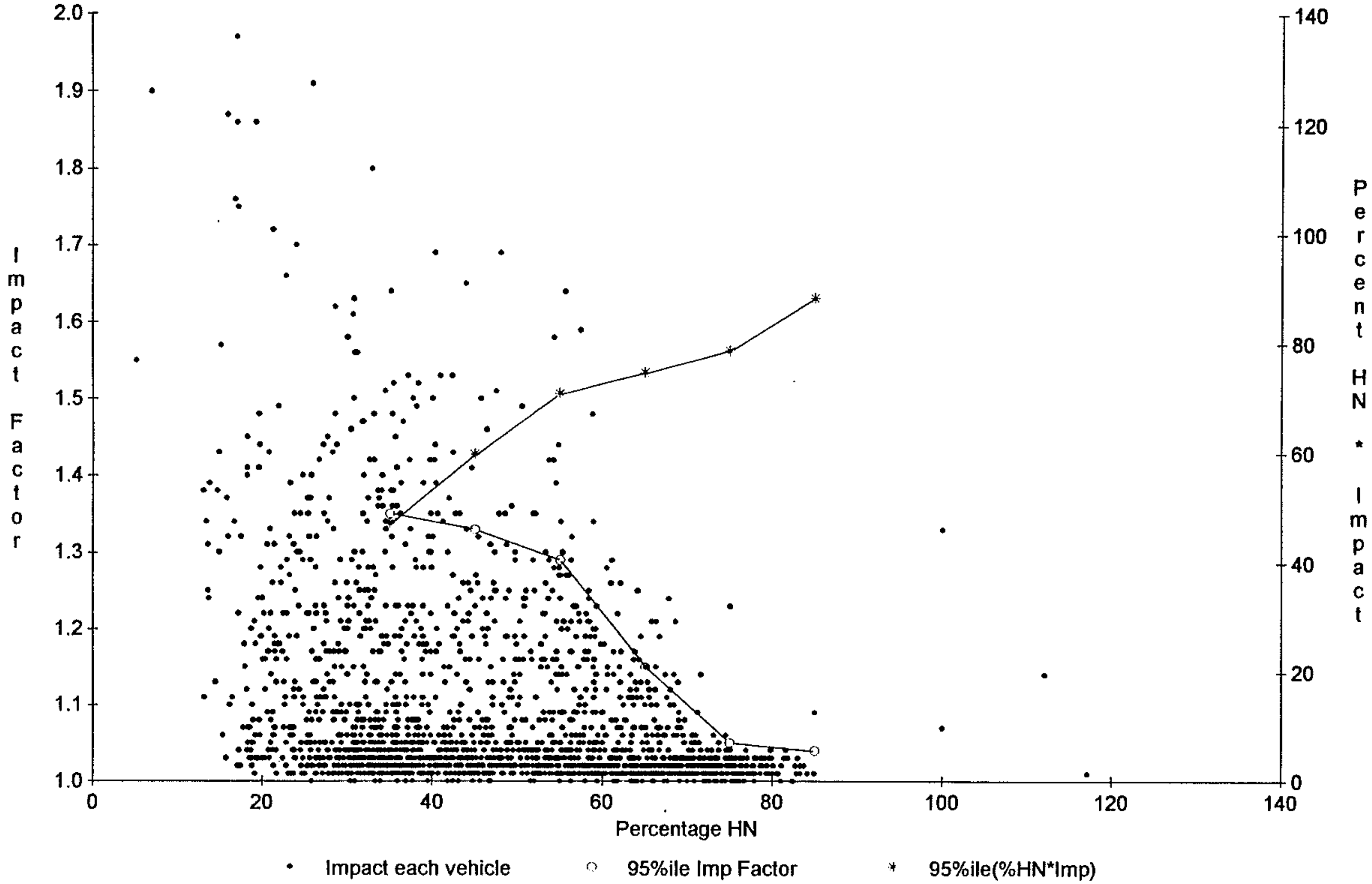


Figure 14: Porewa Stream Br. (Rata)

Plot of all loading events

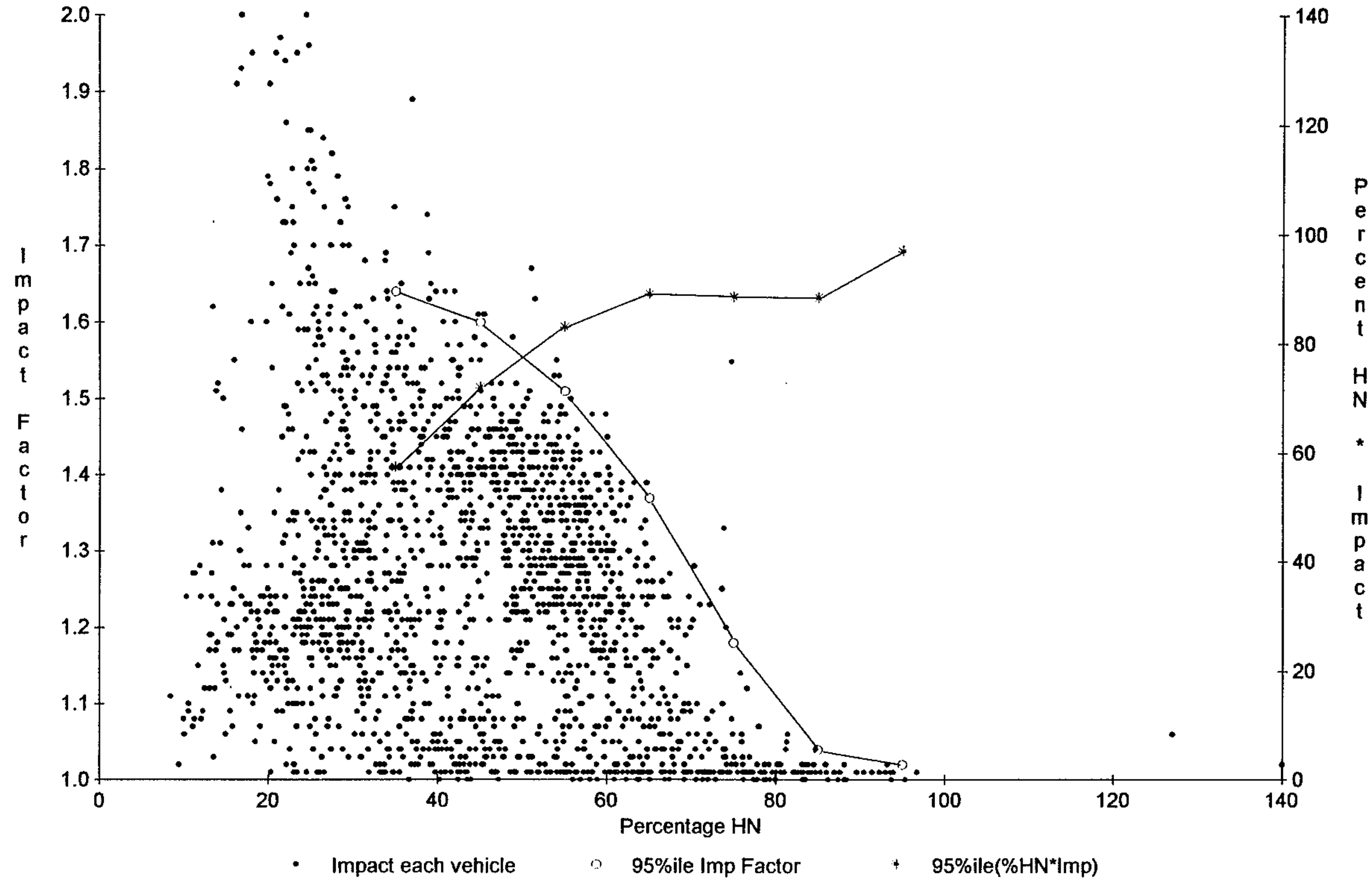


Figure 15: Porewa Stream Br. (Rosses)

Plot of all loading events

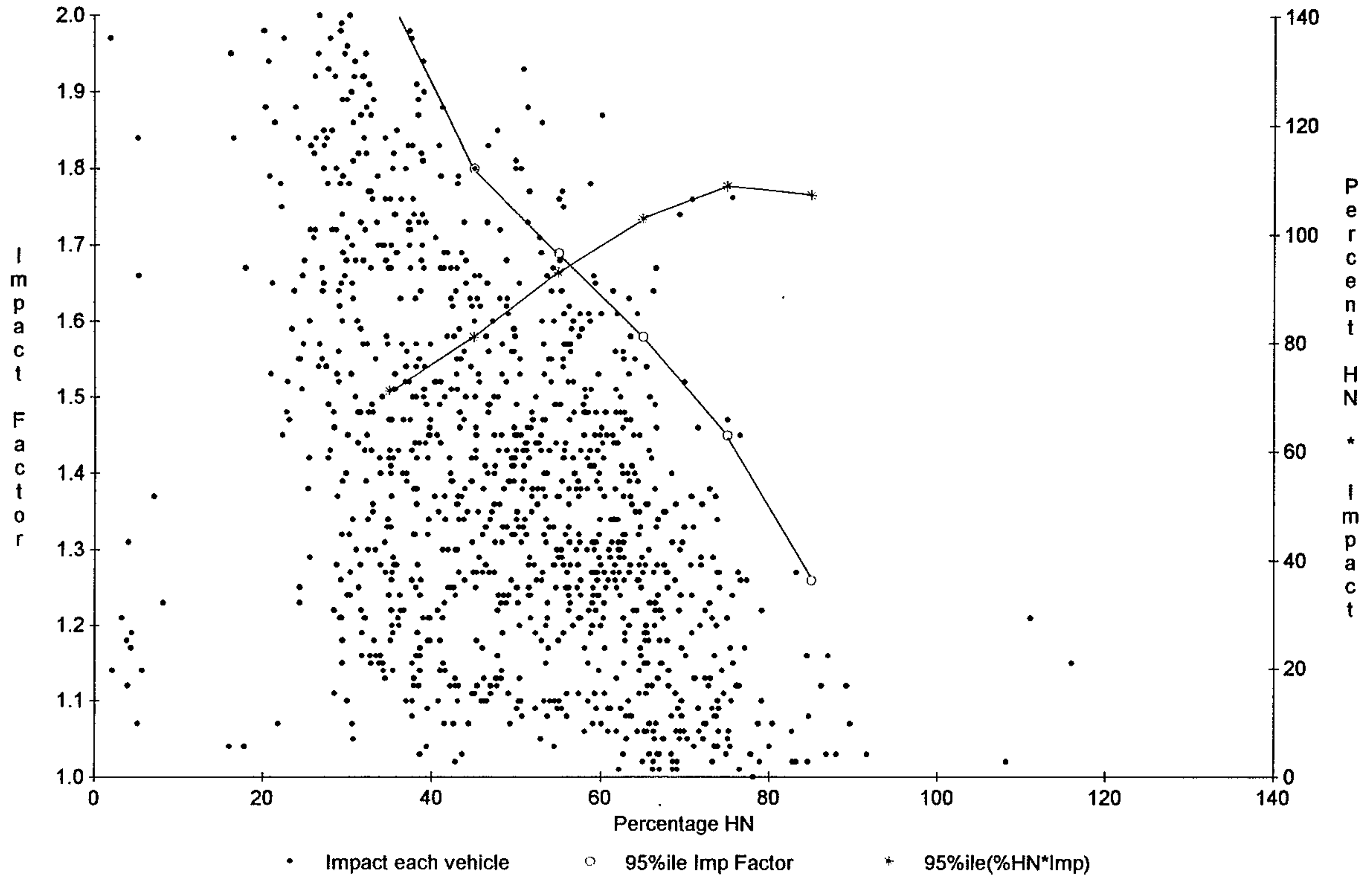
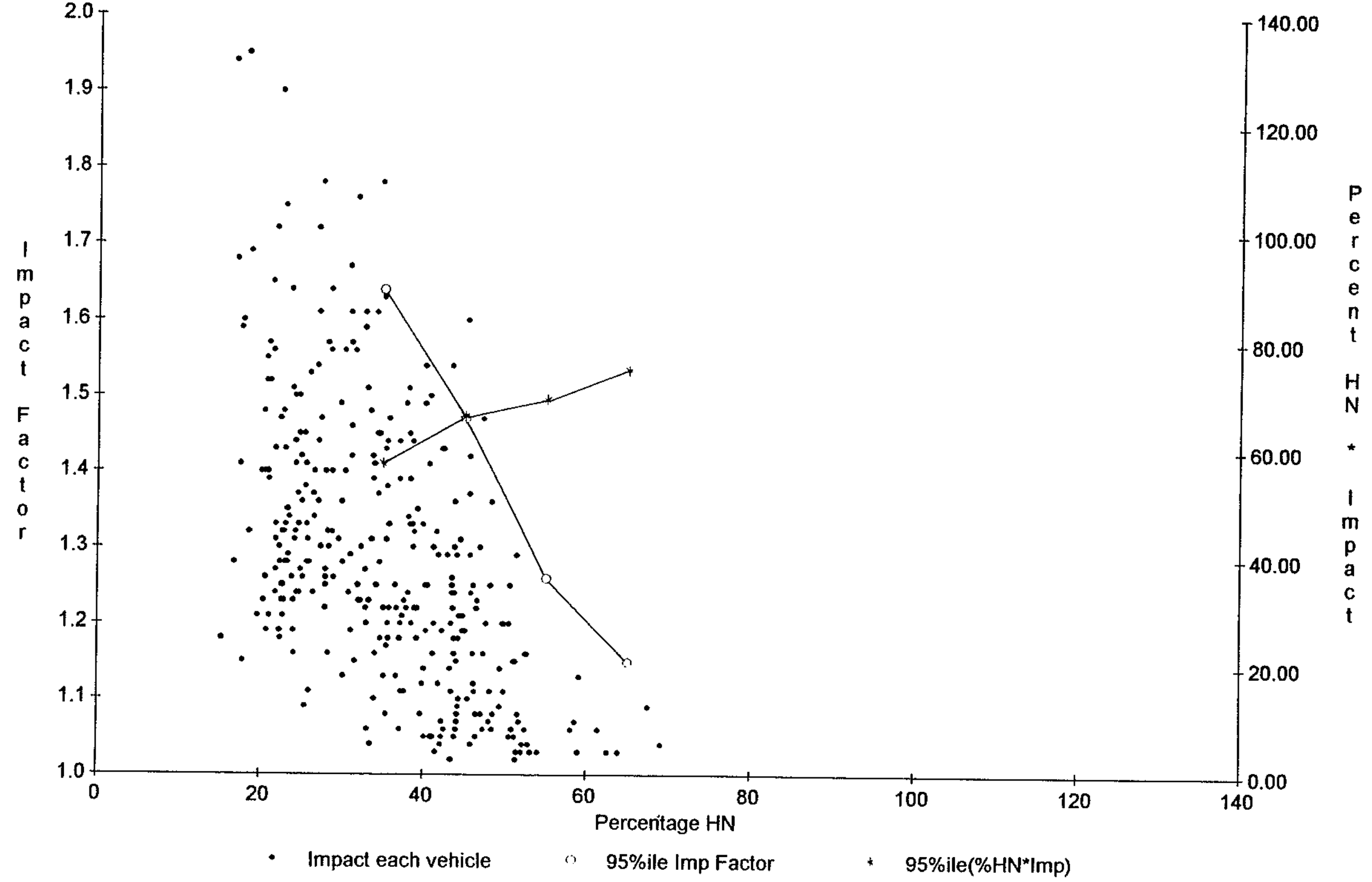


Figure 16: Makohine Stream Bridge

Plot of all loading events



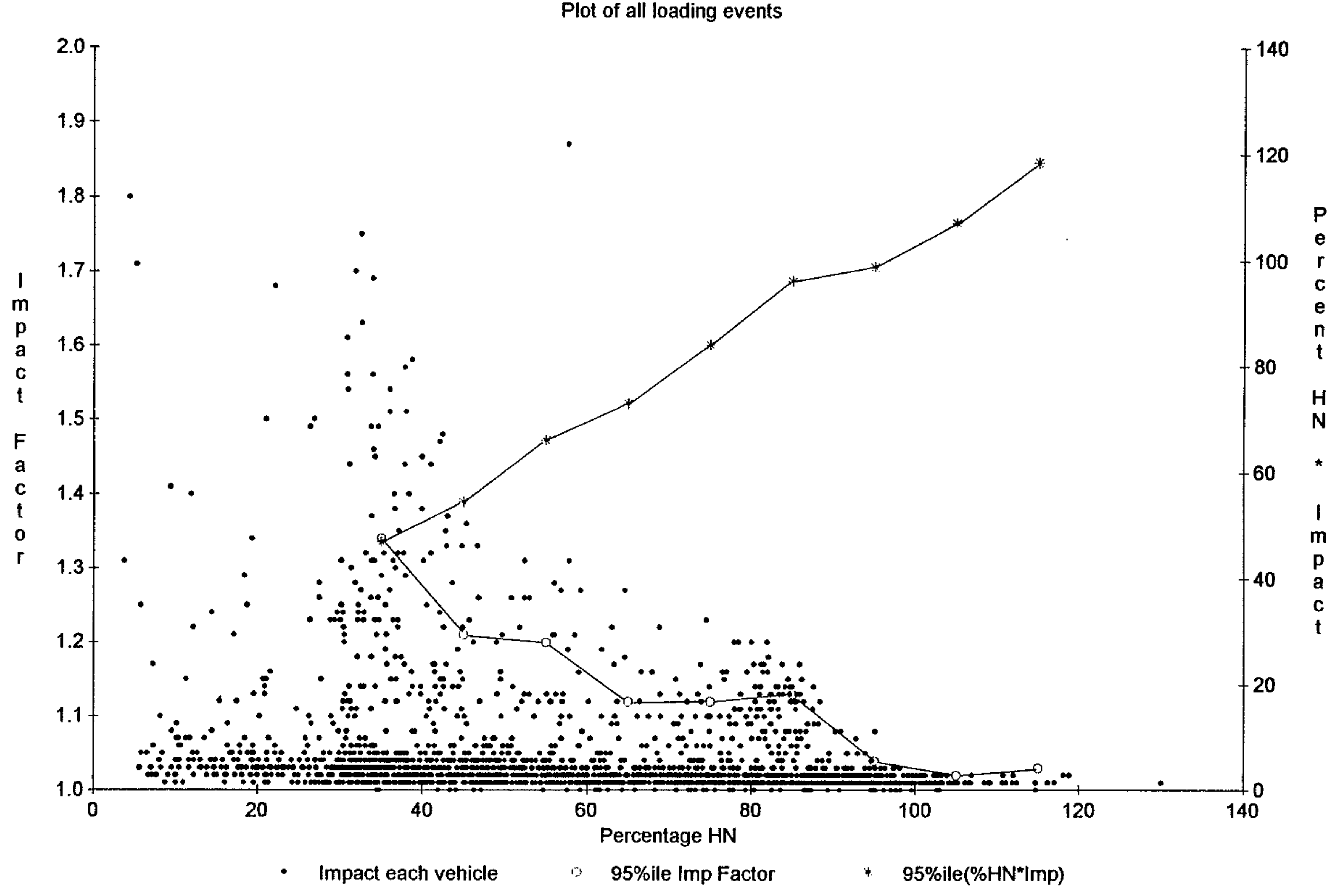


Figure 17: Mangatewai-nui River Bridge

Figure 18: Nat Frequency, Damping Ratio

95%ile I,All Bridges,Both Lanes,85% HN

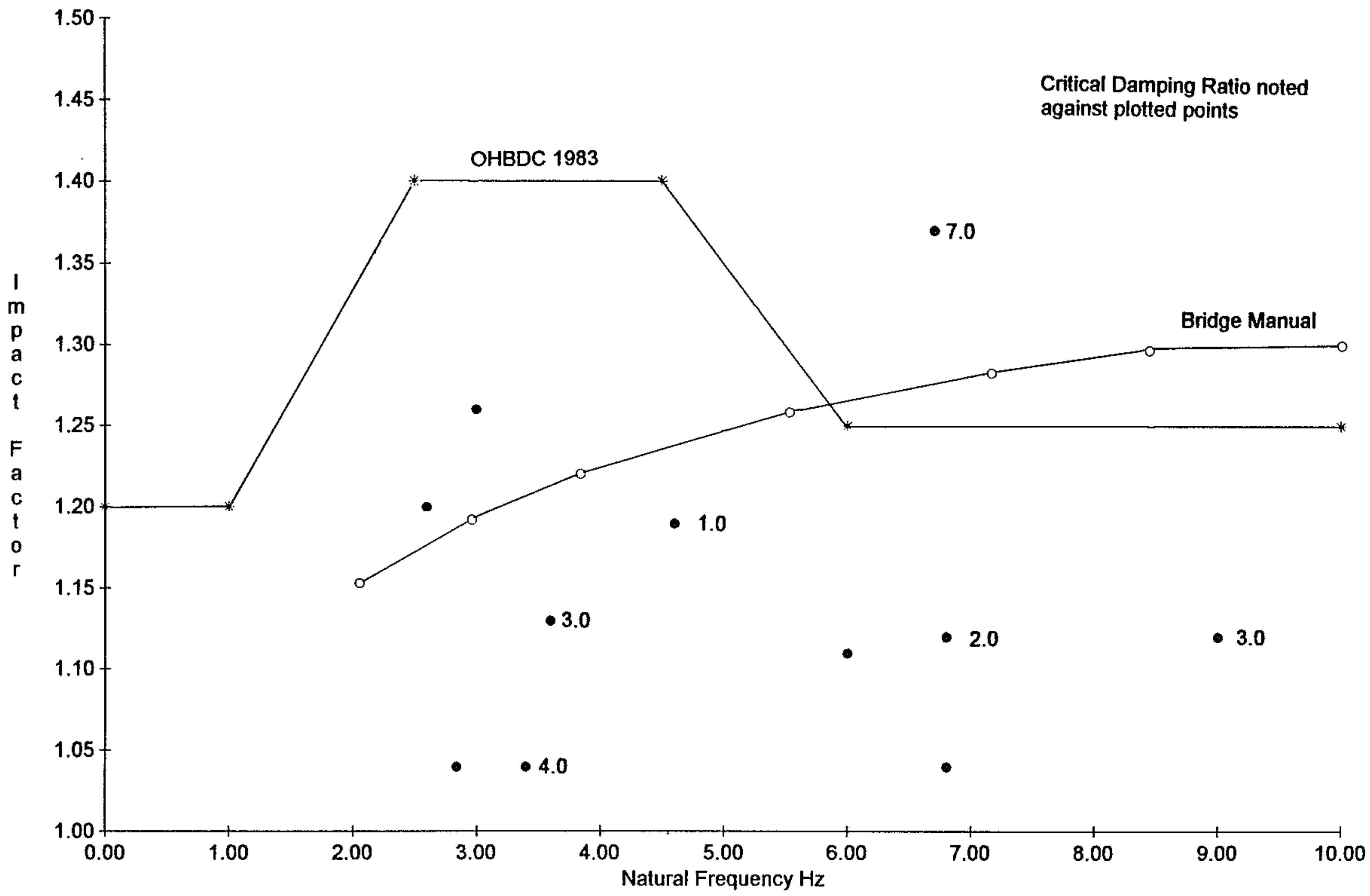


Figure 19: Nat Frequency, Beam Material

95%ile I,All Bridges,Both Lanes,85% HN

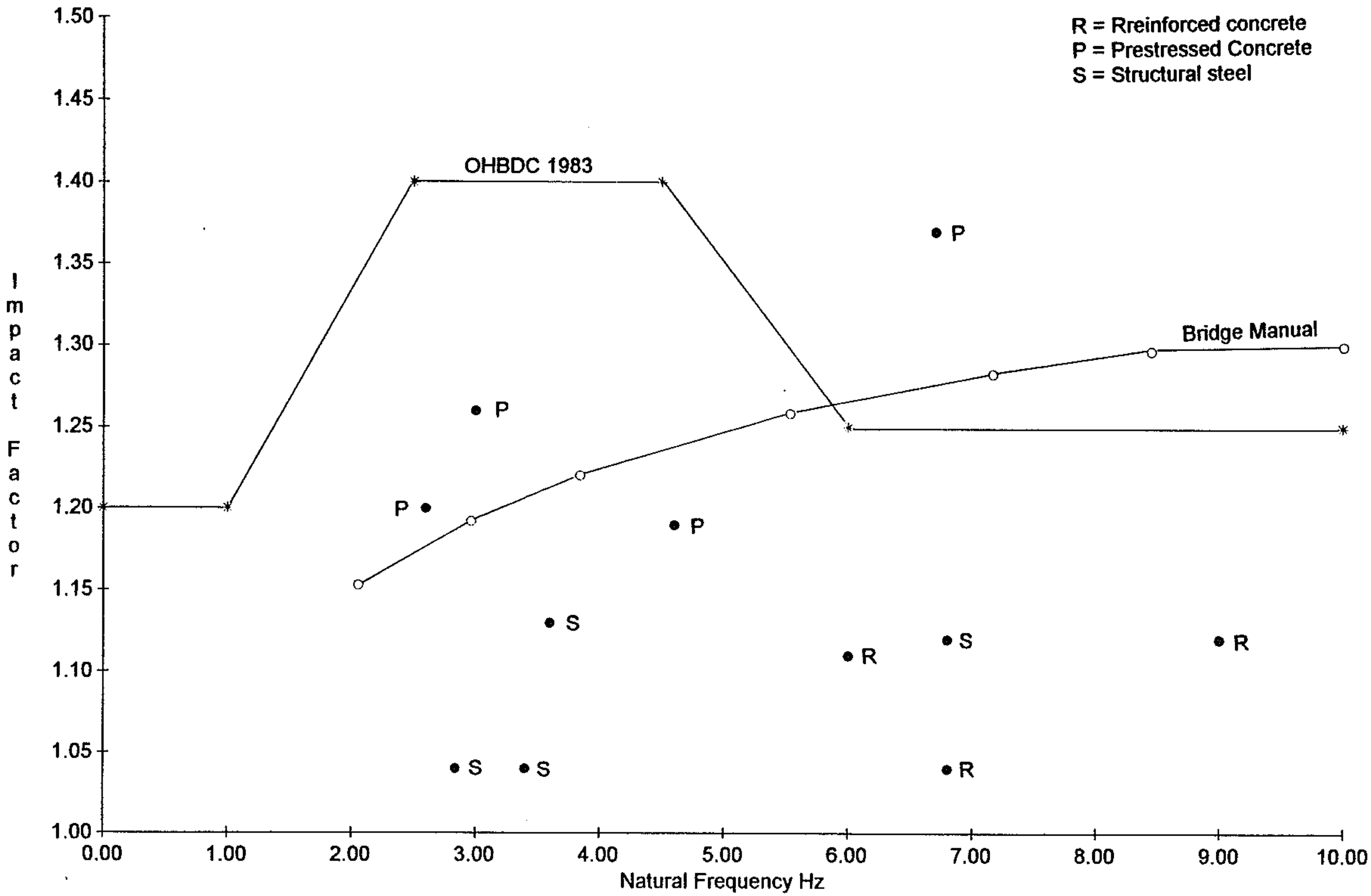


Figure 20: Nat Frequency,End Conditions
95%ile I,All Bridges,Both Lanes,85% HN

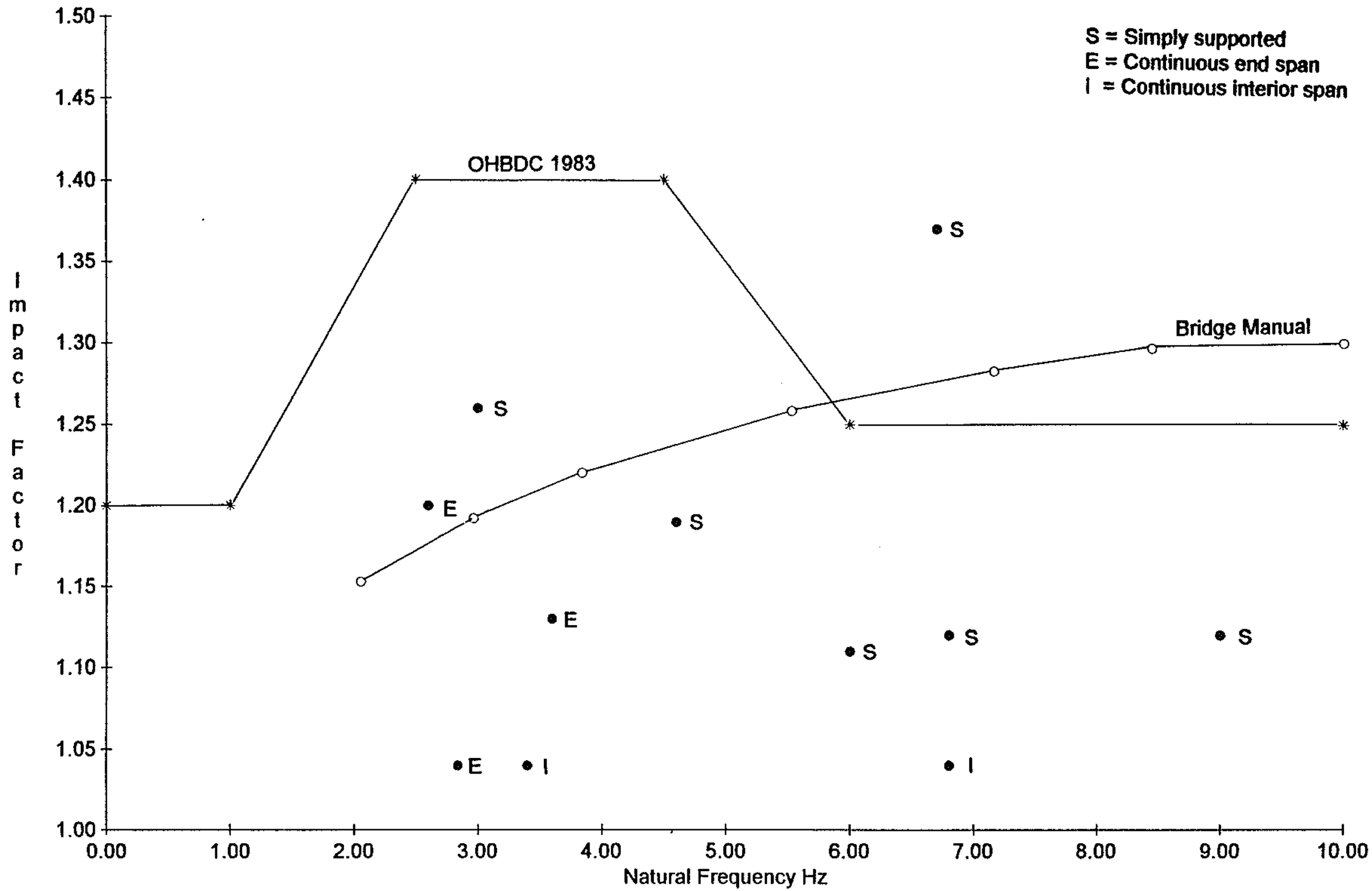


Figure 21: Frequency, Road Roughness

95%ile Impact Factor, Each Lane, 85%HN

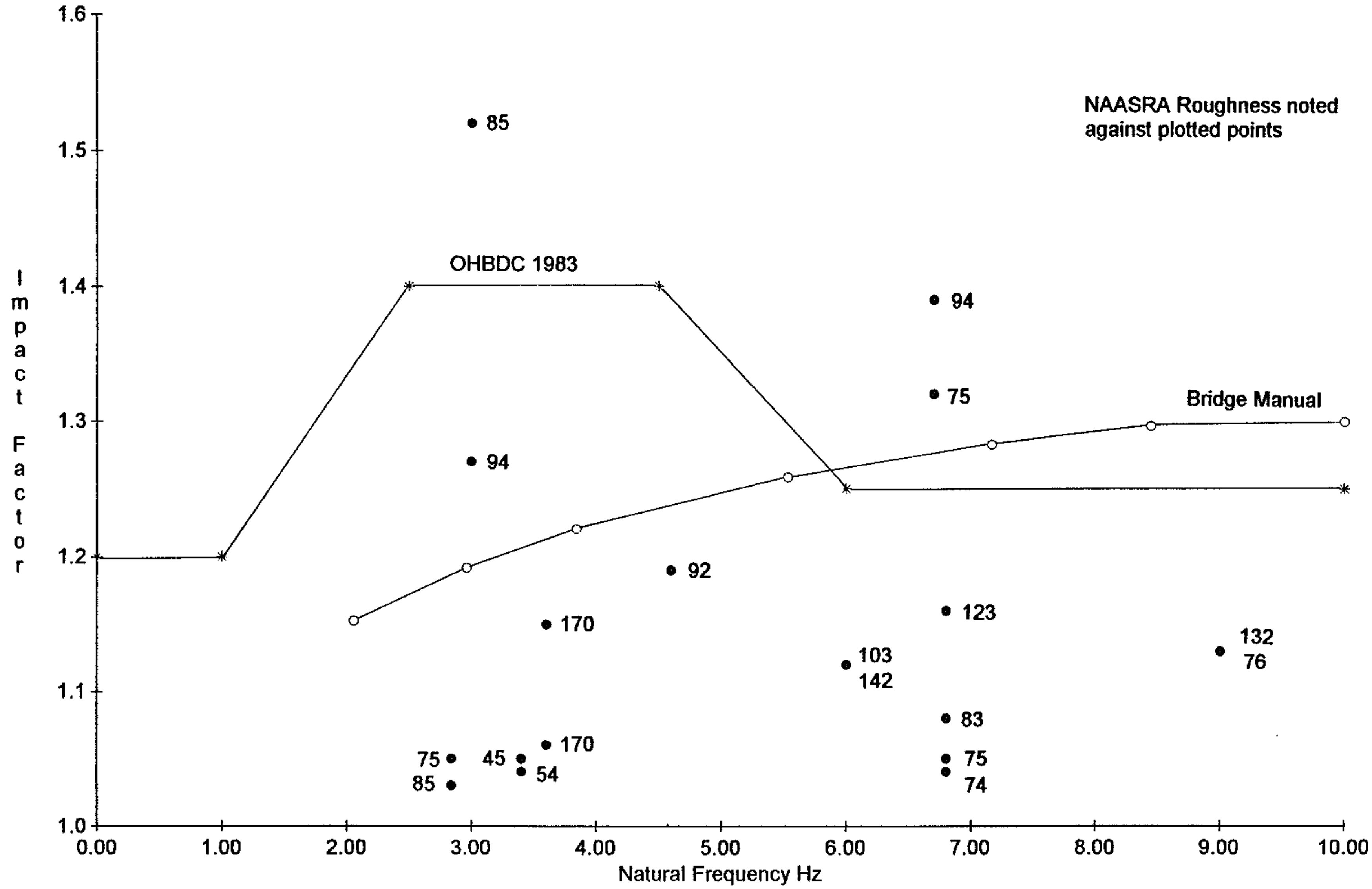


Figure 22: Correlation Against Span

95%ile I All Bridges, Both Lanes, 85%HN

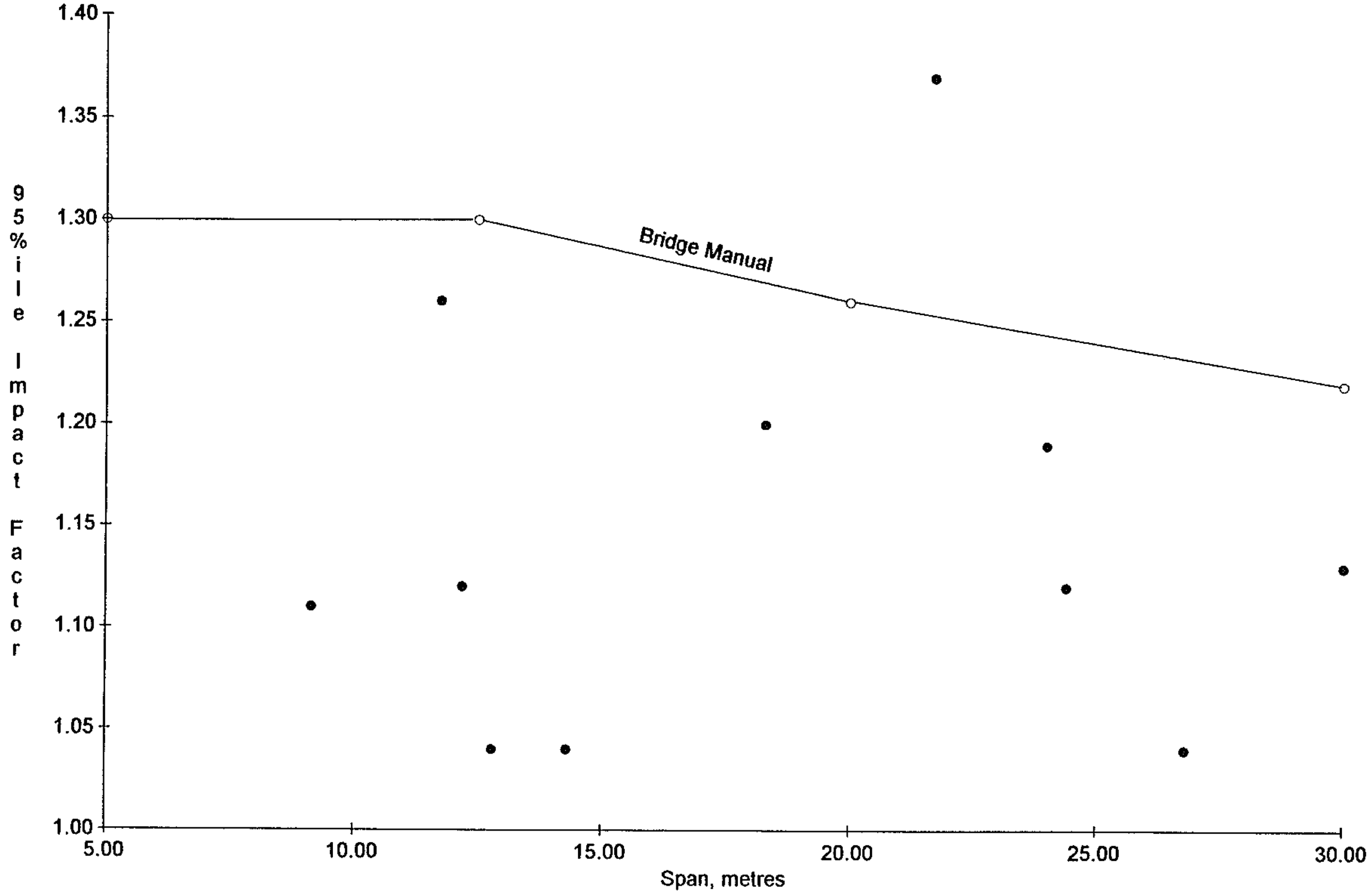


Figure 23: Correlation Against Speed

All bridges, Vehicles 85%HN

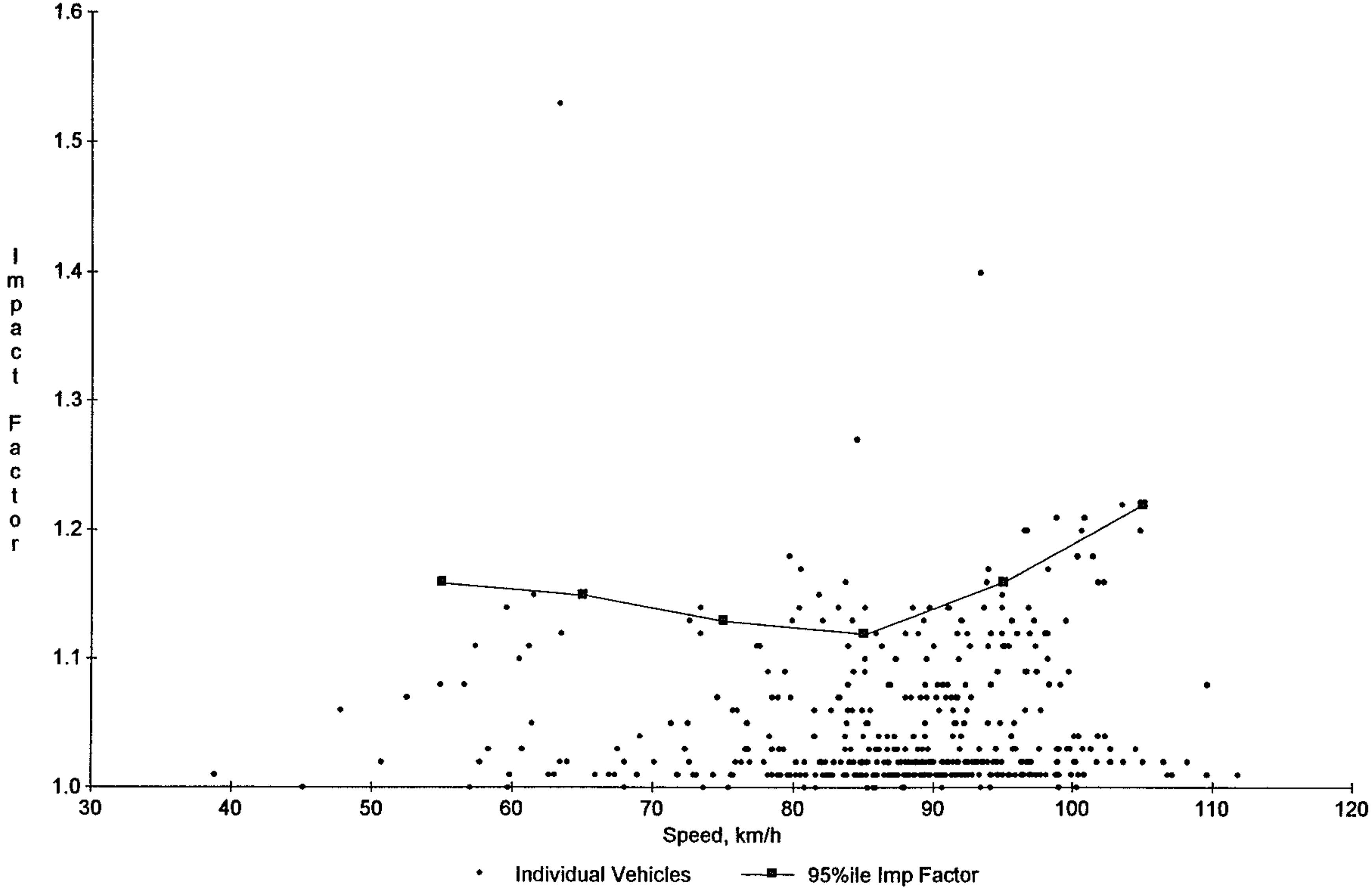


Figure 24: Influence of Load Magnitude

95%ile I, All Bridges, All vehicles

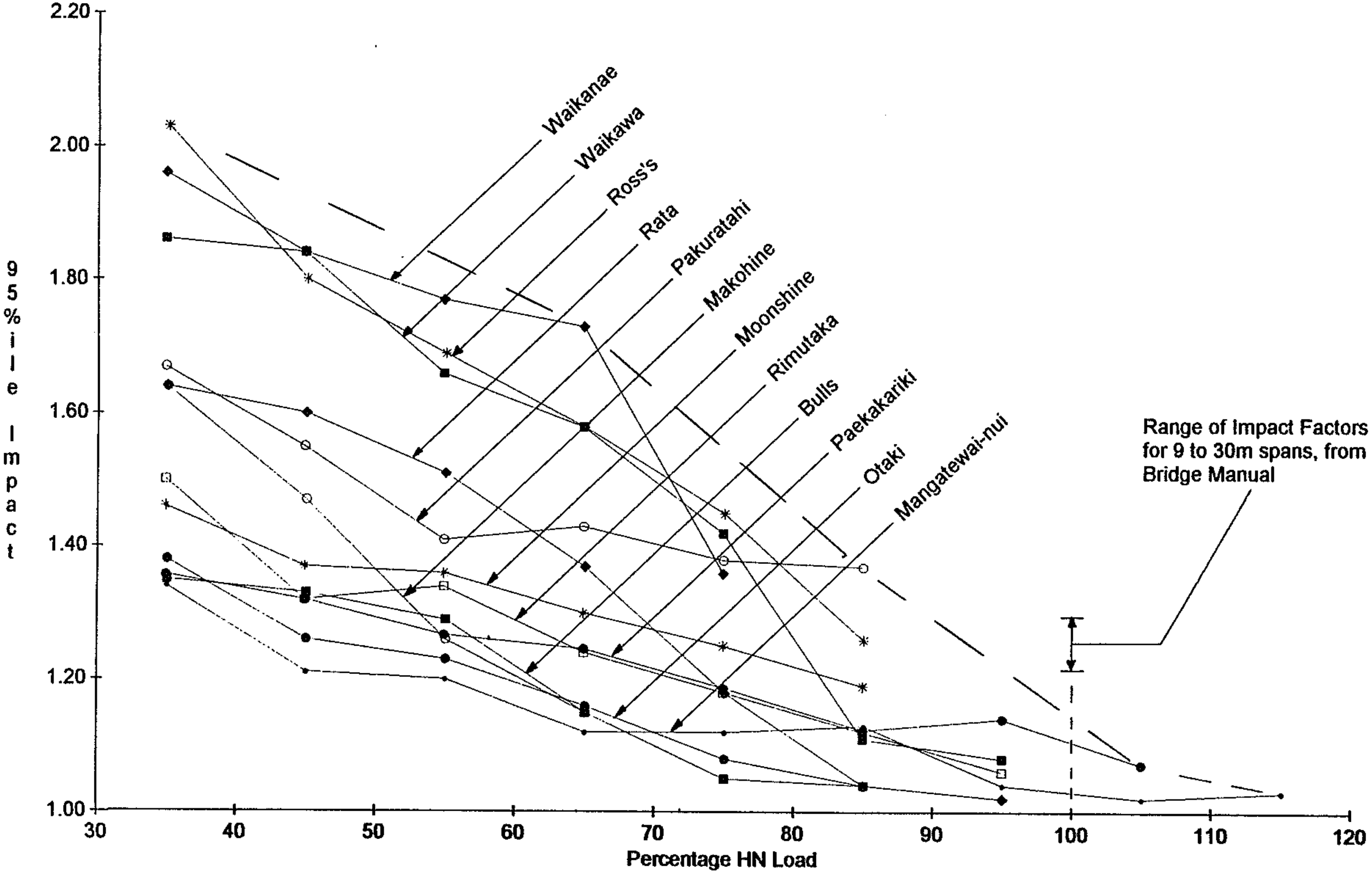


Figure 25: Paekakariki Rail Overbridge

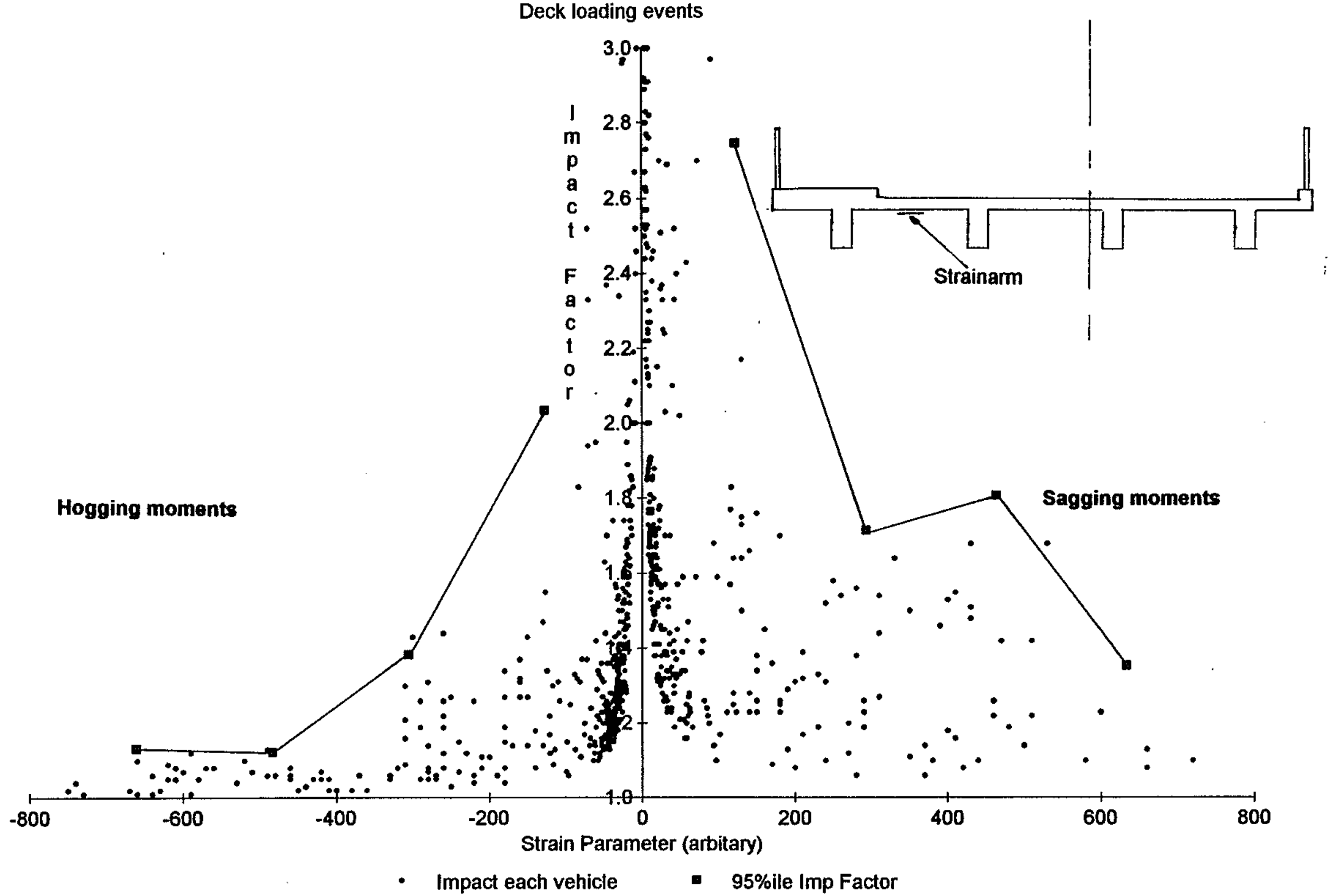


Figure 26: Waikawa River Bridge

Deck loading events

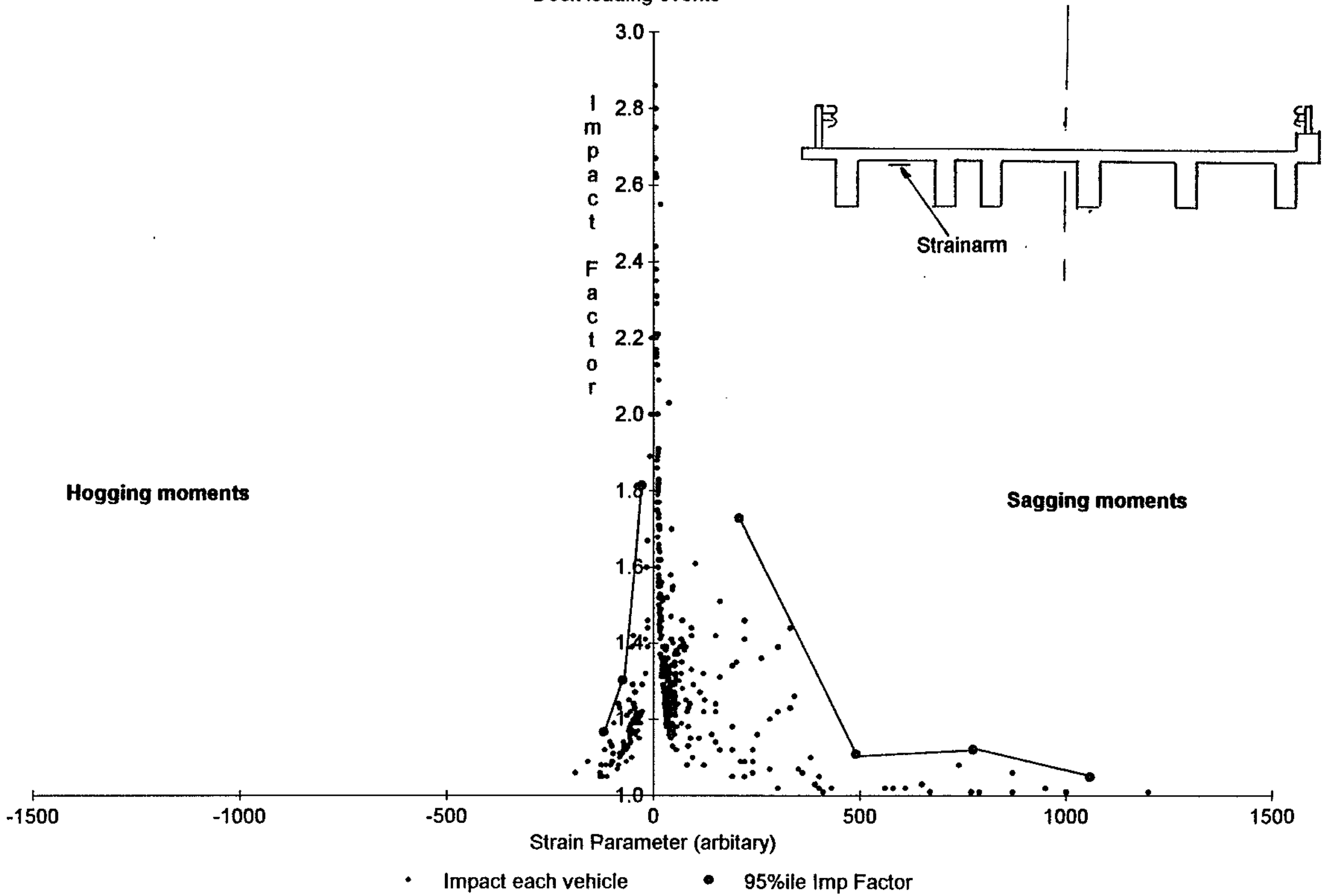


Figure 27: Pakuratahi River Bridge

Deck loading events

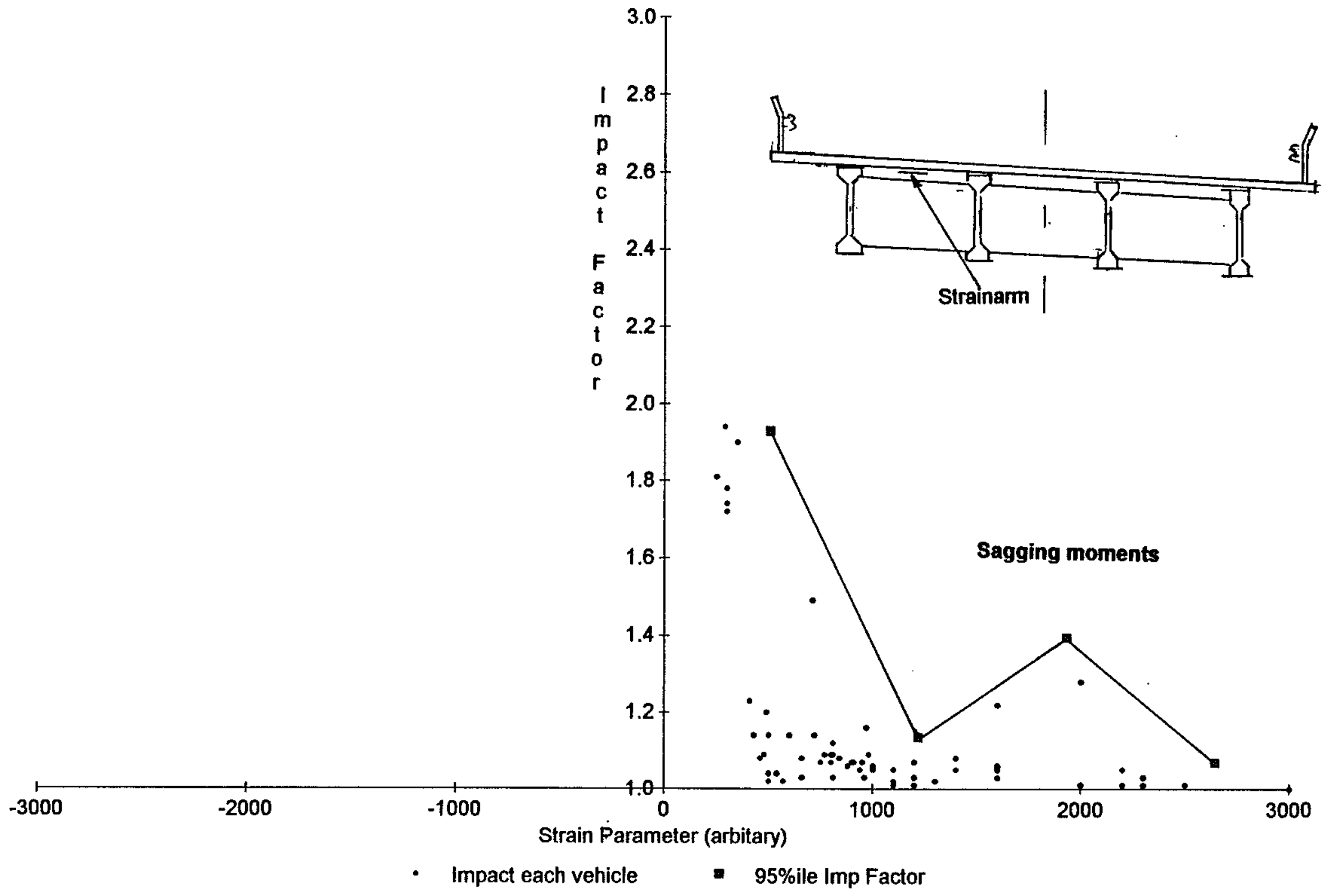


Figure 28: Rimutaka No. 1 Bridge

Deck loading events

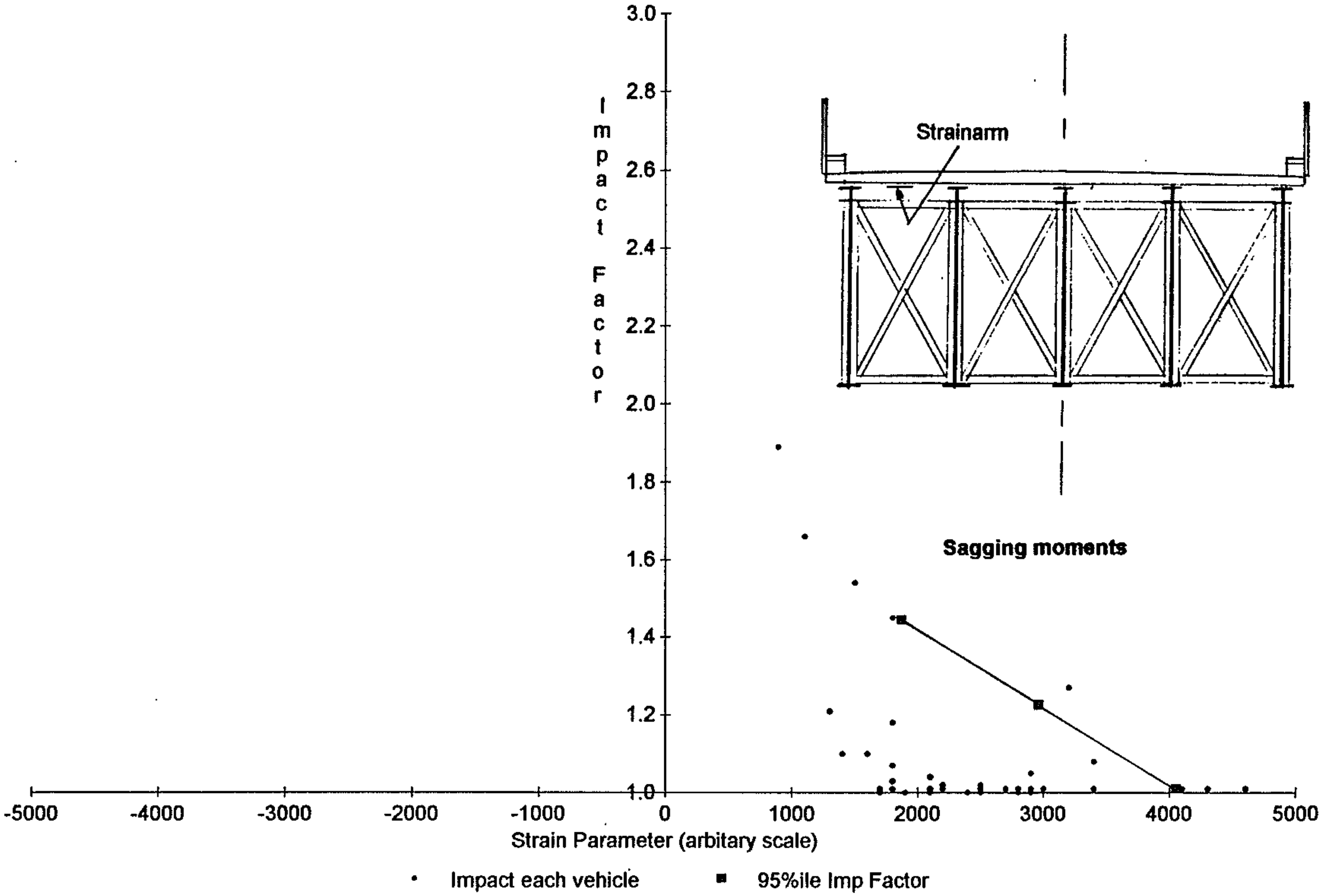


Figure 29: Otaki River Bridge

Deck loading events

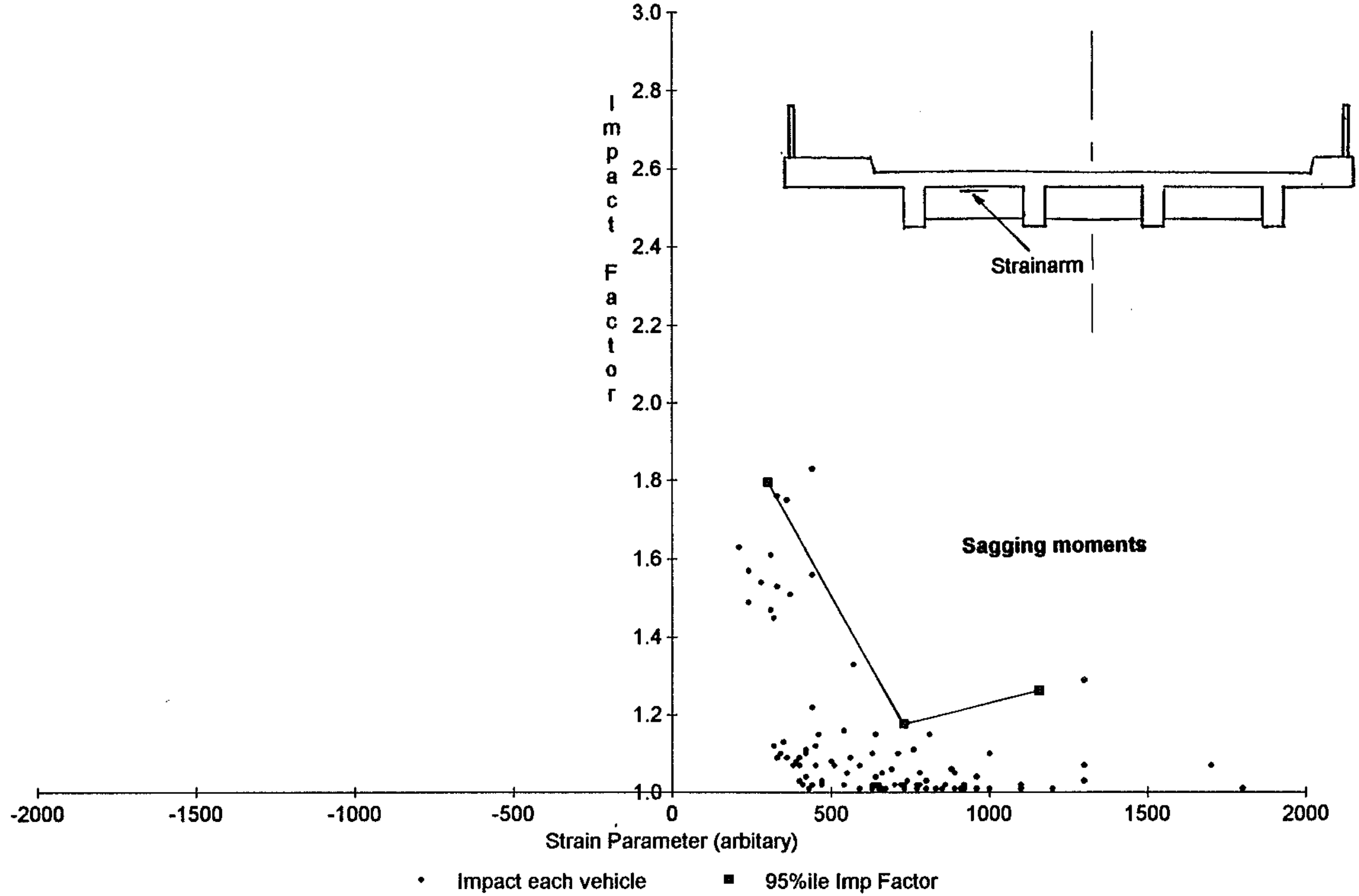


Figure 30: Rangitiki River Br. (Bulls)

Deck loading events

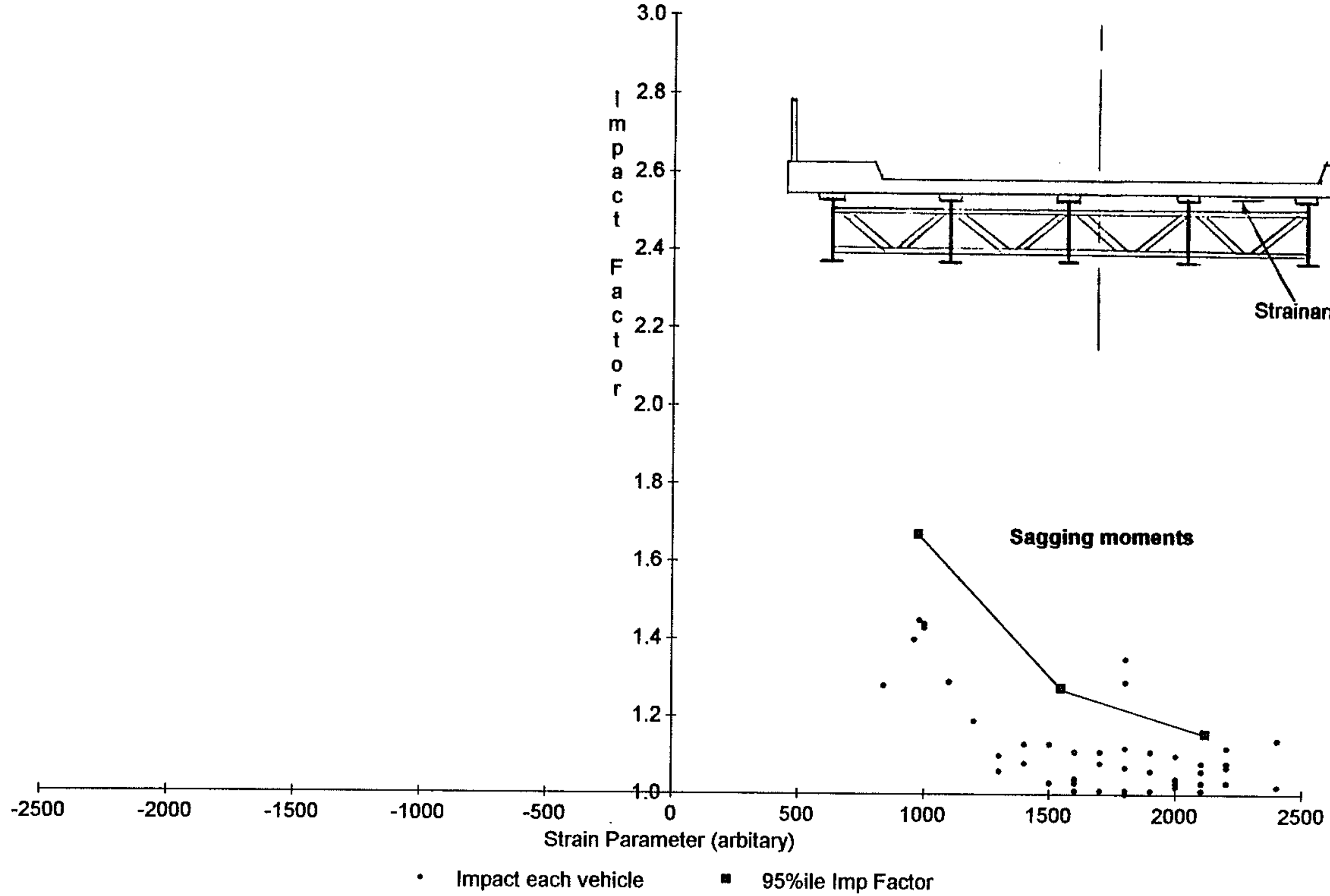


Figure 31: Porewa Stream Br. (Rata)

Deck loading events

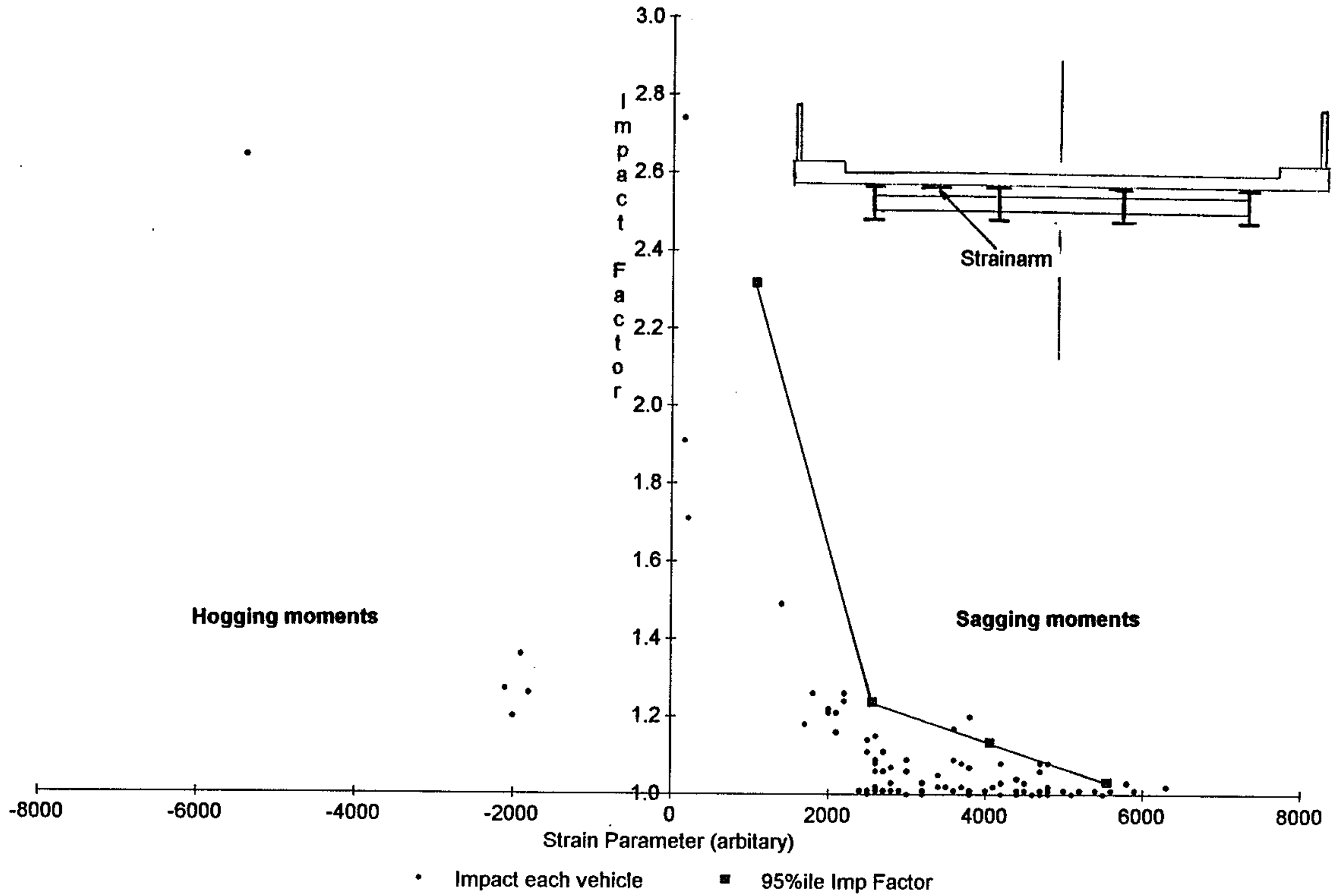


Figure 32: Mangatewai-nui River Bridge

Deck loading events

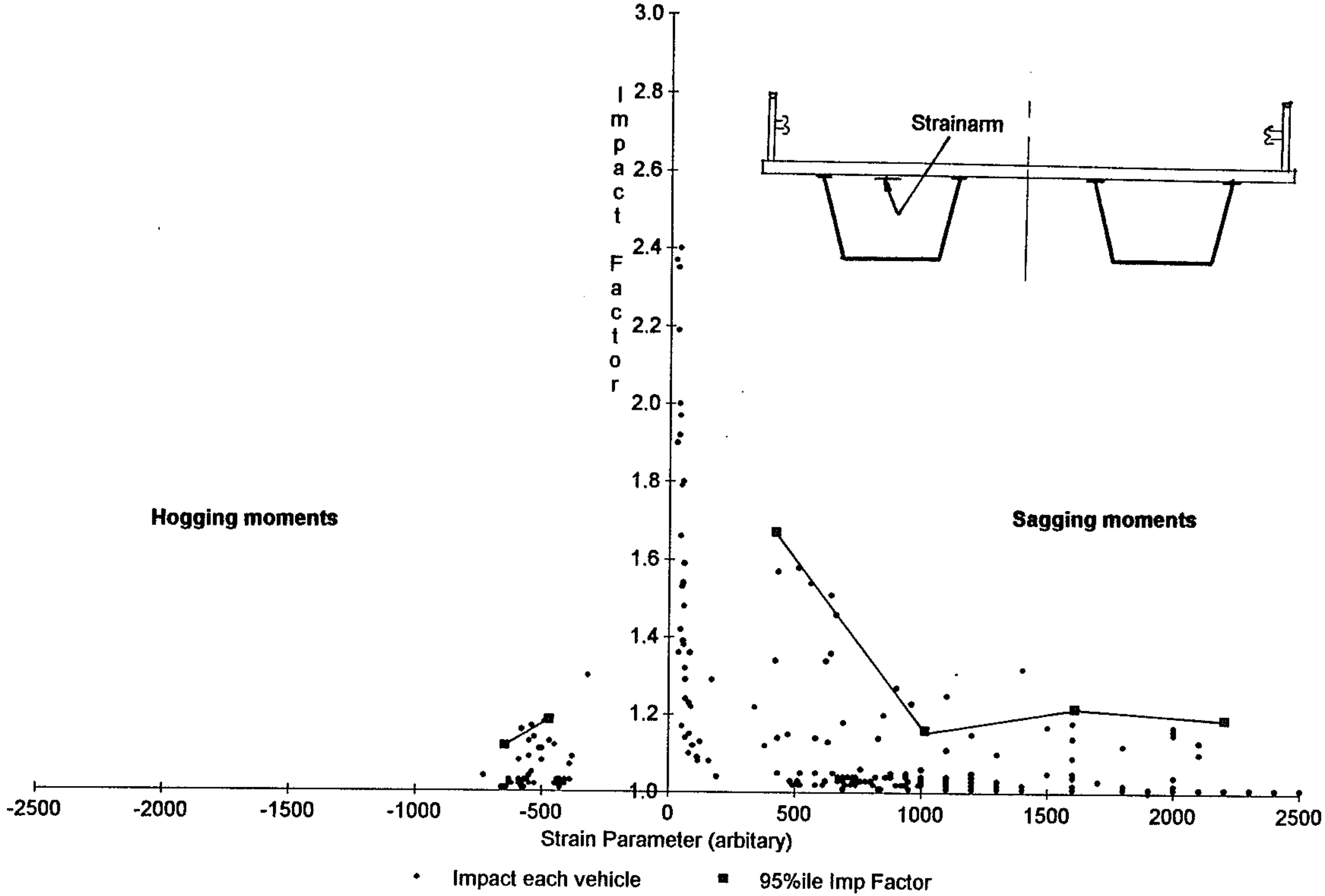
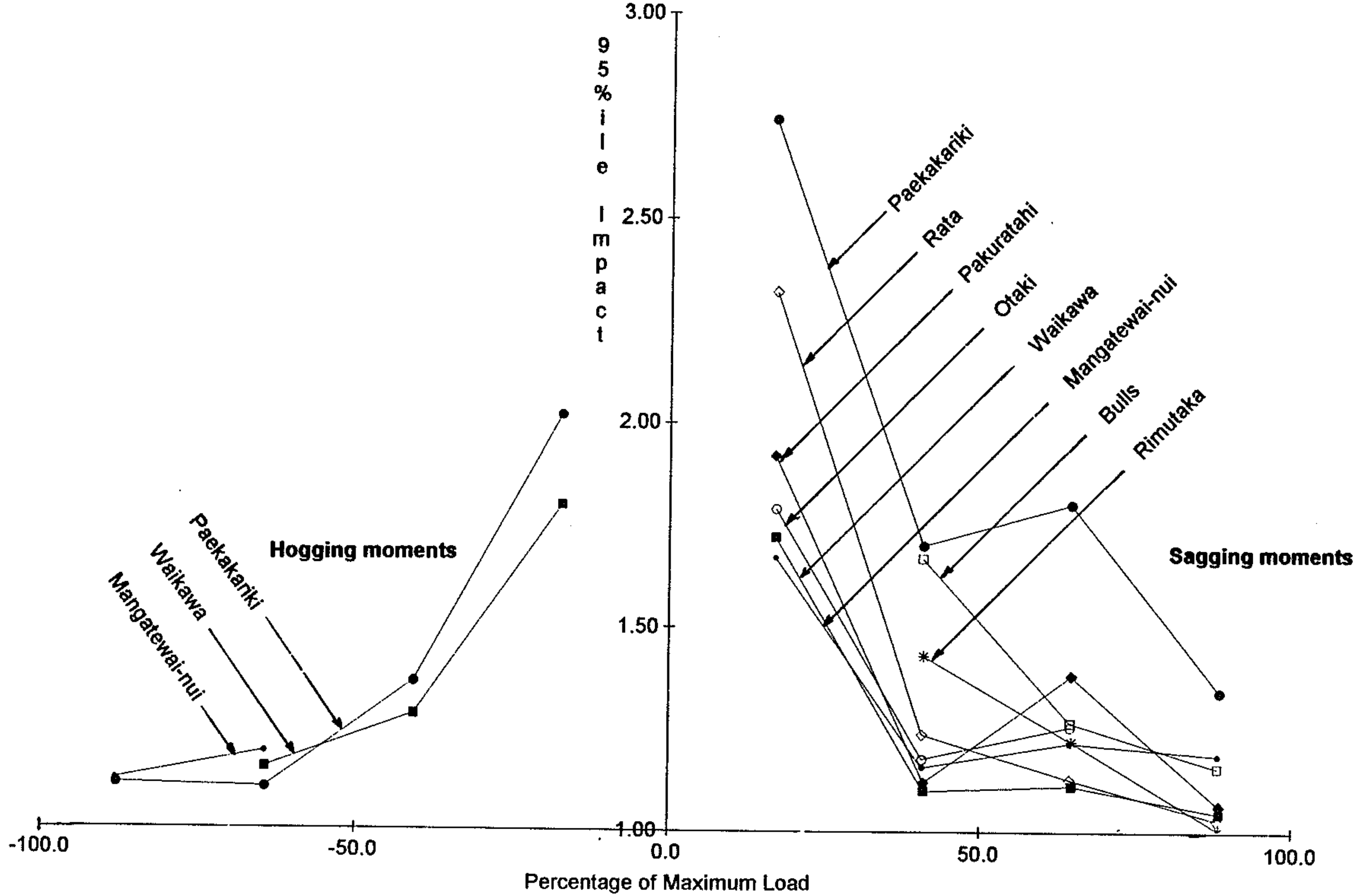


Figure 33: Influence of Load Magnitude

95%ile I, All bridges, Deck loading



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APPENDIX A
SURVEY OF AVAILABLE RECORDING EQUIPMENT

Central Laboratories Report 92-28705.01

SURVEY OF BRIDGE IMPACT LOGGER SYSTEMS

J.A. Gould
Central laboratories, Works Consultancy Services Ltd,
Lower Hutt, NZ
July 1992

CONTENTS

A1.0 Aims
A2.0 Potential Suppliers Contacted

A2.1 Initial Reference List
A2.2 Additional International Research Organisations

A3.0 Results

A3.1 Australia-AXWAY, ARRB and Australian Researchers
A3.2 Transport and Road Research Laboratory, UK
A3.3 Ontario Ministry of Transportation and Communications
A3.4 US Department of Transportation, Federal Highway Administration

A4.0 Conclusions

A5.0 Recommendations

REFERENCES

A1.0 Aims

The aim of this report was to locate and assess the suitability of data logging equipment for studying impact factors on bridges.

A2.0 Potential Suppliers Contacted

From past experience in the data logging field, it was possible to determine that no commercial data logger currently available off the shelf in New Zealand would meet the requirements of this research objective. Hence the search for suitable equipment was directed overseas.

Two avenues were pursued.

A2.1 Initial Reference List

The reference list obtained from P Stanford of relevant research papers was assessed and likely candidates contacted. The assessment was made using the following criteria:

- the reference mentioned physical measurements on actual bridges (many papers were of theoretical computer models);

- the reference was published after 1980. This criterion was set to eliminate obsolete logging techniques as the capability of electronic loggers has expanded by orders of magnitude over the last decade;

- the reference was from an English speaking country. Past experience has indicated that dealing with non-English suppliers incurs considerable extra costs to any project, particularly when the equipment manuals are in a foreign language.

This reduced the list of 26 references to:

- Billing J.R., 1984 (two references)
- Axway System, 1982, 1984 (two references)
- Australian University papers, 1983, 1987 (two references)
- Daniels J.H., 1987 (one reference)

The full references are given at the end of this report.

These were followed up with all but the university paper authors successfully contacted.

A2.2 Additional International Research Organisations

Additional enquiries were made to the Australian Road Research Board (ARRB) and the Transport and Road Research Laboratory (TRRL) of the UK.

A3.0 Results

A3.1 Australia - AXWAY, ARRB and Australian Researchers

These all resolved into the same equipment supplier, namely ARRB. The Axway system indicated in the supplied references was an obsolete ARRB system and has been replaced by their DADAS system which forms the heart of the Culway Weigh-In-Motion (WIM) system that they have supplied to Transit New Zealand. They have supplied these DADAS systems to a researcher in the Department of Transport, Queensland, Mr Geoff Smith. Mr Smith was contacted and his experience of dynamic testing of a single bridge using the modified DADAS WIM system discussed.

Mr Smith's comments were that the equipment performed well for his purpose which was to measure dynamic bridge response for several runs of a test truck. During the test the bridge was closed to other traffic and truck speed was not recorded. His impression was that the DADAS system memory would not contain more than 6-10 full waveform records of test vehicle runs.

When discussing the logger requirements with ARRB it became apparent to both parties that the system software would have to be substantially modified in order to measure the parameters needed for this research. The memory issue is a significant one as the maximum memory available for the DADAS system is 1 Mbyte, this would determine the quantity and quality of data recorded. It would probably limit the system to recording summaries of data rather than raw data and may compromise the precision of the results. The memory limitation may significantly affect the deployment costs also as it may be necessary to retrieve data several times during the test period, Mr Smith suggested daily retrievals would probably be required.

To place this in perspective, if a standard laptop computer were used as the basis of the logger up to 100 Mbyte (100 times the capacity of the DADAS system) of hard disk data storage would be available for on site data recording.

A3.2 Transport and Road Research Laboratory, UK

Mr John Cuninghame of TRRL's Bridge Division was contacted and the logging requirements for this research project discussed. His comment was that TRRL had not done any related research since the 1970's. The only equipment he could suggest was a stress monitor logger called "DynaMonitor" which is currently being used for fatigue life monitoring on the Auckland Harbour Bridge in New Zealand.

The technical support people for this equipment in Auckland were contacted and the logging requirements of the bridge impact logger discussed. The DynaMonitor bought for the Auckland Harbour Bridge is still fully committed to ongoing monitoring. The cost of the system when bought was $NZ60,000 (20,000 Pounds Sterling) and has almost doubled in price since then to $NZ114,000. The system is a 6 channel recorder and does not measure vehicle speed (a requirement of this research project). The price, the unavailability of the existing unit, and its technical limitations make the DynaMonitor logger impractical for this research project.

A3.3 Ontario Ministry of Transportation and Communications

The research leader into bridge dynamics, Dr J R Billing, was absent on vacation until early August. Contact was made with his associate Mr A Agarwal, a structural research engineer who has been involved in the work that led to the two papers by Dr Billing and subsequent work.

As described in their paper (Billing, 1984) these researchers used the Ontario MOT&C Instrument and Testing Section's mobile data logging facilities which are housed in a truck. The equipment described (Billing, 1984) has undergone several upgrades and currently is computer based, using a data acquisition system. This equipment truck is heavily utilised by the Ontario MOT&C staff and being a universal tool for all their field logging is overly complex (and expensive) for the requirements of this research project. Mr Agarwal did indicate that storing whole waveform recordings and analysing them later enabled the researchers to precisely identify all the relevant effects. Some bridges in their studies produced uncharacteristic results and these unexpected effects would have not been easily identified with logging equipment which simply recorded impact factor ratios.

Mr Agarwal was most helpful in relating their experiences and techniques. He also said Dr Billing would provide advice on data analysis programs and techniques. Keeping in contact and discussing progress with this group of leading researchers would be a significant advantage for this project.

A3.4 US Department of Transportation, Federal Highway Administration

This followed the source of a paper (Daniels, 1987) describing a "WIM+RESPONSE system capable of acquiring and processing data to provide information on bridge dynamic effects".

After some time a Mr H R Bosch was located who, while not the major author of the paper, was involved in the work and now holds the position of senior research engineer in the Structures Division of the US Department of Transportation, Federal Highway Administration, Turner-Fairbank Highway Research Center in McLean, Virginia. Mr Bosch said the basis of the system described in their paper (Daniels, 1987) was originally a bridge WIM system using the bridge deflections to weigh trucks. After substantial modifications, both hardware and software, it was developed into a dynamic effects logger for that project. His comments indicated that there was a significant development effort involved.

Subsequently a simplified, low cost version of this "Bridge WIM System" has been marketed commercially by the Toledo Scales Company. This simplified system has limited capability. Although the system developed (Daniels, 1987) is still currently used by some of the staff at the research centre and other WIM systems have been adapted for other projects, they are one off specialised loggers, not available for general use.

Mr Bosch said the research staff involved in dynamic bridge measurements no longer follow the approach modifying WIM equipment and are currently using IBM PC laptop class computers with data acquisition systems. The staff tend to use either commercial logging software or, because of limitations of the commercial packages, write their own logging software. Mr Bosch's preference was to develop his own software and he considers this a very cost effective approach. He also made the comment that adapting a WIM system was not as flexible as using a laptop.

A4.0 Conclusions

Of the six promising leads, none had off the shelf equipment suitable for this research project. The closest equipment to the project's requirements of the six contacts is the DADAS system supplied by ARRB.

The base cost of the ARRB DADAS hardware is $NZ19,400 ($AU14,000). The additional cost of specialised software to configure it for the requirements of this research project is estimated to be approximately $NZ15,000, which would exceed the budget.

Central Laboratories' experience with WIM systems has shown that fixed program loggers of this type require significant refinement before they work reliably. With an overseas supplier, significant effort has to be undertaken with each problem in determining and documenting the cause and effects. In addition. the turnaround in getting program updates to fix a given problem usually takes weeks. In the case of WIM stations, our experience is that these issues can go on for months or even years. There is a significant level of risk that these issues would compromise the research effort.

In contrast, if the design staff are local and a conventional laptop computer is used for logging, the turnaround for problems would be reduced by at least an order of magnitude.

The two leading overseas researchers (Billing, Canada and Bosch, USA) currently working in this field both adopt the IBM type computer plus data acquisition system approach, and one (Bosch) has specifically discarded the converted WIM loggers.

A5.0 Recommendations

This equipment survey has determined that the most cost effective approach to obtaining a logger for this research project is to continue with the third stage of developing a logger locally. This logger should use the same hardware approach as Bosch (USA) and similar bridge instrumentation to Billing (Canada).

It is recommended that the third stage proceeds with these guidelines.

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APPENDIX B
DESCRIPTION OF RECORDING EQUIPMENT DEVELOPED

RECORDING EQUIPMENT DEVELOPED FOR THE PROJECT

The basic sensing device employed was a strainarm having a gauge length of 300mm, designed to be fastened to the surface of concrete or steel bridge members. The design of the strainarm was based on those employed on the Culway weigh-in-motion system developed by the Australian Road Research Board. The shape of the device at its central point creates approximately a tenfold strain amplification. Four strain gauges are positioned to record tensile and compressive strains at the central point of the strainarm, and are wired as a full Wheatstone bridge. The strainarms were attached to either beam or slab soffits as appropriate for each bridge, and the output from each was individually amplified before input to the recording device. Strain measurement was adopted in preference to displacement, because strainarms would be less susceptible to vandalism than displacement detection devices which would need to be connected to the ground.

In order to trigger the recording, and to enable vehicle speed to be computed, pairs of pressure sensitive strips were attached to the road surface on the approach to the span being investigated, in the two directions. These sensors were a known distance apart, nominally 10m, in the direction of traffic movement.

The recording device was a laptop computer, with a 60Mb hard disk, running MS-DOS. The two sets of input channels were interfaced into the centronics port, with capacity for 8 straingauge channels and 8 traffic sensor channels, although only 4 of the latter were used. The programming included a self-monitoring re-booting facility which would operate if the system crashed. The power supply was two 12 volt, 60 Ah batteries, which were estimated to be enough for two weeks operation, or approximately 2000 loading events.

The system remained in standby mode until activated by the leading axle of a vehicle crossing one of the traffic sensors, at which time scanning of all straingauge channels started, at intervals of 0.0077 seconds. Scanning continued until 3 seconds after the last axle of the vehicle passed the sensor, or until a time limit was reached. The limit was intended to allow a closely following convoy of about 10 vehicles travelling at 80 km/h through as one event. Also recorded was the time each axle crossed each sensor. The strain input channels were not re-zeroed to avoid errors due to temperature effects, but zeroing was performed on each strain record during later analysis. The incoming data was scanned to determine the maximum strain level for each event, and if this was less than a value corresponding to about twice that produced by a typical car, the event data was deleted. Accepted data was initially stored in RAM, in blocks of about 40 vehicles or about 1 Mb, and then passed to the hard disc.

In order to obtain data on the behaviour of deck slabs, which have a very much higher natural frequency range, the scanning frequency was increased ninefold between the hours of 12 noon and 2 pm, and the deck slab channel only was scanned.

A separate calibration program allowed an on-screen plot to be made of strains recorded by individual vehicles against time, so that the sensitivity of the strain gauges, and the trip level could be determined at the calibration stage.