# Assessment of the Historic Seismic Performance of the New Zealand Highway Bridge Stock

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This paper presents the assessment of historic seismic bridge performance of the New Zealand highway bridge stock from the 1968 Inangahua earthquake through to the 2016 Kaikōura earthquake. Spatial ground motion details based on recorded and observed ground motion intensities were used to estimate the peak ground acceleration (PGA), as a measure of the seismic demand at each bridge location. Across all events, a PGA of 0.05g or higher was experienced on over 800 occasions across bridge sites. Damage characteristics were collated from available literature, with the majority of the highway bridges experiencing either no damage or only minor damage across all the events. At PGAs greater than 0.5g the number of bridges with moderate and major damage was still relatively small. There was also no clear differences between the performance of bridges across the different design eras, despite the varied design and construction practices. Some shorter bridges may have performed better than expected due to the effect of abutment stiffness and damping, while some longer bridges may have performed well due to travelling wave effects. These findings will inform future assessment methods and design, and the accuracy of analytical modelling of the bridge stock.

Keywords: bridges, seismic performance, seismic assessment, fragility, New Zealand, historic earthquakes, network vulnerability

## 1 Introduction

New Zealand is a seismically active country, and as such, the effects of earthquakes on infrastructure can be significant. New Zealand transportation networks have little or no redundancy throughout the country. With the reliance on transport networks for essential services such as fast-moving consumer good delivery, it is critical to ensure the network remains functional after a seismic event (Davies et al., 2017). Bridges are a key part of the road network, and yet there are currently many unknowns related to the actual seismic response of the bridges across New Zealand and internationally. Having a good understanding of bridge performance during earthquakes is essential to practising engineers, authorities managing the bridges, and decision-makers who need to make important retrofit priority and post-earthquake serviceability decisions to ensure the usability of the bridges and safety of the public. The objective of this study is to assess the performance of highway bridges in historic earthquakes in New Zealand and to develop a dataset that can be used to test the applicability of bridge assessment methods and analytical modelling approaches.

The performance of bridges in recent earthquakes has been reported in a number of studies, including events in Japan (Bruneau, 1998; Watanabe et al., 1998), Taiwan (Chang et al., 2000), the United States (Basöz & Kiremidjian, 1998; Basöz et al., 1999; Hwang et al., 2000; Wang & Lee, 2009), Iran (Eshgi & Ahari, 2005), Peru (Taucer et al., 2009), China (Wang & Lee, 2009), Italy (Kawashima et al., 2010), Chile (Schexnayder et al., 2014), and New Zealand (Mason et al., 2017; Palermo et al. 2010; Palermo et al. 2011; Palermo et al. 2017; Wotherspoon et al., 2011). The potential performance of bridges in future earthquakes has commonly been characterized through the use of fragility functions (Billah & Alam, 2015; Pan et al., 2010; Sung et al., 2013; Tavares et al., 2012).

These fragility functions can broadly be categorized as those that were developed based upon observed seismic damage (empirical), and those that were developed primarily through analytical models and simulation (analytical). Fragility functions developed based upon observed damage data include those for bridges in Japan and Greece. Fragility curves based on data from the 1995 Kobe earthquake were constructed by using empirical methods, with variation of input ground motions and structural parameters (Karim & Yamazaki, 2003; Yamazaki et al., 2000). In Greece, analytical approaches were initially used, before being calibrated against empirical curves based on damage data from the US and Japan (Basöz et al., 1999), due to the absence of corresponding data from European earthquakes (Moschonas et al., 2009). A wider range of studies have developed analytical fragility curves that were not able to make use of case history data to validate their models. Examples include studies based on large datasets of bridges from Korea (Lee et al., 2007), Italy (Borzi et al., 2015) and the United States (Gidaris et al., 2017).

It is clear from past research that the majority of methods used for assessment of the seismic performance of bridge stocks internationally have not been validated against case history performance. This paper aims to collate case histories of the performance of the bridges on the New Zealand State Highway network from the 1968 Inangahua earthquake through to the 2016 Kaikōura earthquake. The general sense from the research and practising engineering community was that most bridges had performed better than expected in these recent earthquakes in New Zealand, and there is a desire to know if the observed performance was in line with the expected risk profile of the bridge stock (Wood et. al., 2017). In order to investigate the expected performance of the bridge stock, the development of the seismic bridge design philosophy in New Zealand and the characteristics of highway bridge stock is first presented. Then, the method for estimating the seismic demand in past earthquakes at each bridge site and the collation of evidence of bridge damage during each event is discussed. As the New Zealand State Highway bridge stock could be different from other parts of the world, especially in terms of typologies and design standards, the applicability of the internationally developed fragility functions in New Zealand is also examined and discussed. Lastly, performance in past events was compared against estimated performance based on national scale high-level seismic screening of the bridge stock.

## 2 Development of Seismic Bridge Design in New Zealand

For most of the twentieth century, bridge design and construction in New Zealand was controlled by a single centralised organization operating under the various names of the Public Works Department (PWD), Ministry of Works (MoW), or Ministry of Works and Development (MWD), until it was privatised in 1988, resulting in the majority of the bridge stock in New Zealand being designed and built by a single entity. The New Zealand Transport Agency (NZ Transport Agency) currently manages the operation of the State Highway network. Seismic bridge design standards have been developing along with the changes in the organizations controlling the design and construction. These standards, published by NZ Transport Agency and its preceding organizations, defined the requirements for traffic, wind, flood, temperature and seismic loading. Requirements for member design and detailing of various materials were either described or referenced to the appropriate material Standard. The change in design standards can be described in two aspects: seismic loading and detailing requirements. Based on these changes, the development of bridge design standards in New Zealand can be classified into six eras as shown in Table 1 (Hogan et al., 2013).

**Table 1. Development of Bridge Design Standards in New Zealand (after Hogan et al. 2013)**

| Era | Years | Standards | Number of Bridges | % of total stock |
|---|---|---|---|---|
| 1 | pre-1930s | No Seismic Standards | 212 | 7.8% |
| 2 | 1930s to mid-1960s | Early Seismic Standards | 1338 | 49.5% |
| 3 | mid-1960s to mid-1970s | Preliminary Ductile Standards | 368 | 13.6% |
| 4 | mid-1970s to late 1980s | Early Ductile Standards | 293 | 10.8% |
| 5 | late-1980s to early-2000s | Basis of Current Standards | 185 | 6.8% |
| 6 | early-2000s to – Present | Current Standards | 305 | 11.3% |

Era 1 refers to the period where no seismic provisions were in place in New Zealand, and bridges built during this era are assumed to have no specific design or detailing for seismic actions. In Era 2, early seismic standards and elastic design were introduced. Working stress method was employed in this era where design stresses were to be kept below allowable stress defined for a given failure mode. Bridges were required to be designed to resist a lateral force of 0.1 times the weight of the superstructure, with no variation to account for ground conditions or seismic hazard. Some initial detailing requirements were introduced during this era. Preliminary ductile standards were introduced in Era 3, along with preliminary guidelines on capacity-based design. Based on the improved understanding of seismic hazard, three seismic zones were defined and seismic coefficients dependent upon fundamental period of the bridge were introduced. In Era 4, the use of capacity-based design principles became the standard approach. Era 5 forms the basis of current design standards. The design spectra were converted from a single inelastic spectrum for each zone with an assumed ductility of six, to an inelastic design spectrum for each level of ductility. Performance criteria, seismic detailing, and clauses relating to liquefaction and lateral spreading were introduced. Additionally, site classification to account for different soil sites were introduced. In Era 6 subsoil conditions were further developed to three and five classes in 2003 and 2004, respectively (Hogan et al., 2013). The development of these standards has governed the seismic design of bridges within each era and may affect their performance in historic earthquakes. Therefore, these eras will be referred to throughout the paper in relation to this historic performance.

## 3 Overview of the New Zealand Highway Bridge Stock

There are approximately 2700 highway bridges on the State Highway network in New Zealand that are managed by the NZ Transport Agency. Figure 1 shows the distribution of these bridges across New Zealand as of November 2017 (New Zealand Transport Agency, 2017).

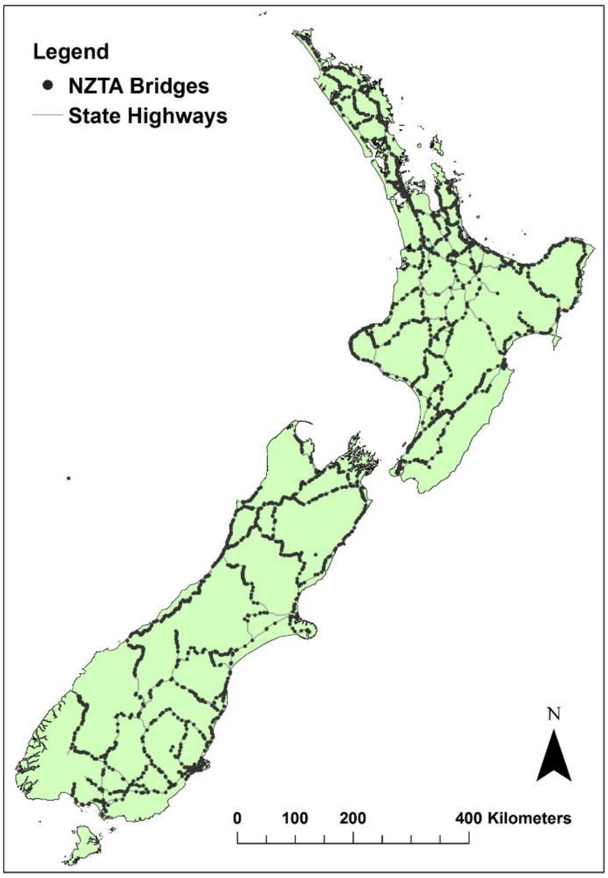


**Figure 1. Map of New Zealand with overview of the State Highways and bridge stock**

Figure 2 summarises the distribution of superstructure types for State Highway bridges built after 1900. Cast-in-situ concrete bridges were by far the most common superstructure construction method before the mid-1950's, which was linked to integral bridge construction. The use of precast concrete superstructures started to become popular after the mid-1950's. Precast concrete superstructures in this study include pre-tensioned superstructures, and the increase in the use of precast concrete is due to the advent of pre-stressing in the 1950's. Figure 3 shows the distribution of bridge length for State Highway Bridges built after 1900. Bridges up to 50 m in length are by far the most common in the bridge stock. Approximately 40% of the bridge stock is single span, with more than two-thirds of the bridge stock having three spans or less. Figure 4 shows the distribution of bridge foundation for State Highway

Bridges built after 1900. Deep foundations are the most commonly used pile type due to over 80% of State Highway Bridges crossing some form of waterway. These deep foundations include driven concrete piles, which include precast piles with either mild steel or pre-stressed reinforcement, and driven steel casings with concrete infill. As compared to deep foundations, shallow foundations were used minimally, mostly for single span bridges, to reduce foundation costs (Hogan, 2014). Figure 5 shows the distribution of bridges based on pier type. Most of the bridges in the bridge stock have no piers, having only abutments, as most of the bridge stock consist of single span bridges. For bridges with piers, reinforced concrete walls represent by far the most common pier type in the bridge stock. Based on these construction trends, a large number of bridges in New Zealand are similar and regular in form, dominated by cast-in-situ concrete bridges, with driven concrete piles and having three spans or less. Further analysis of these datasets are presented in Hogan (2014).

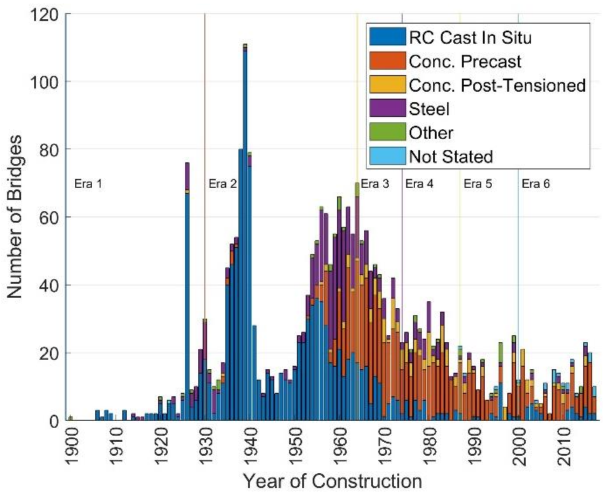


**Figure 2. Distribution of superstructure types for State Highway bridges built after 1900**

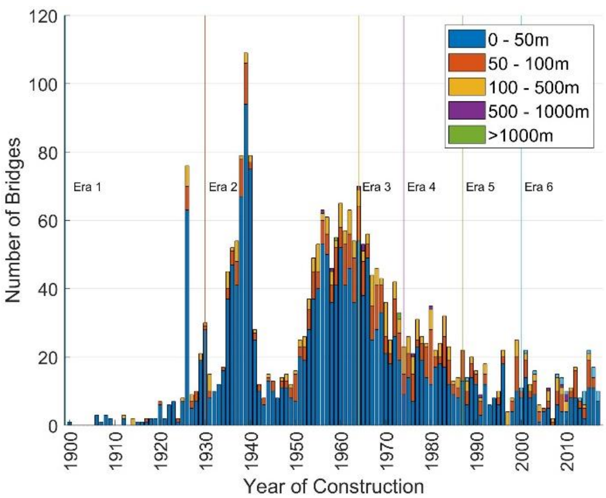


**Figure 3. Distribution of the lengths of State Highway bridges built after 1900**

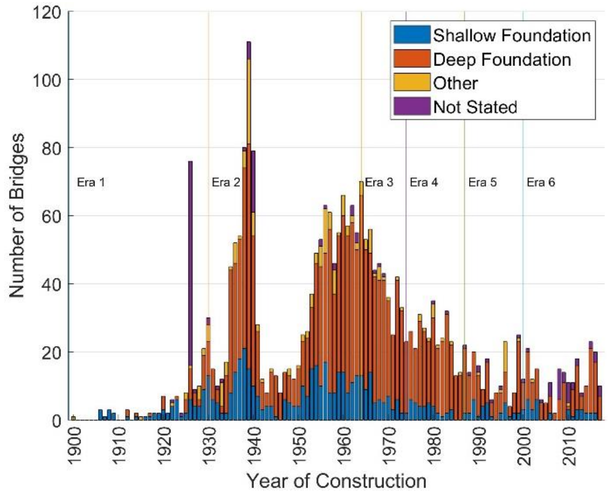


**Figure 4. Distribution of foundation types for State Highway bridges built after 1900**

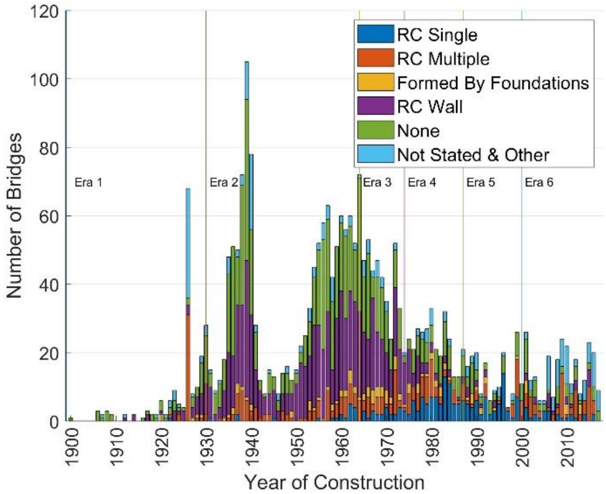


**Figure 5. Distribution of pier types for State Highway bridges built after 1900**

## 4 Bridge Stock Assessment Methodology

The performance of bridges has been assessed and reported after major historic earthquakes in New Zealand, however there has not been a systematic collation of bridge seismic demand and performance across these earthquakes and in other recent earthquakes in New Zealand. In order to assess the historic seismic performance of the bridge stock across a range of earthquakes, three main steps were undertaken. First, to characterise the seismic demand at each bridge location, the peak ground acceleration (PGA) was estimated through geostatistical interpolation of recorded and felt data. Next, the bridge damage characteristics were defined using available literature and were classified into damage severities related to structural and geotechnical damage. Lastly, the estimated performance of each bridge based on a national scale high-level seismic assessment of the bridge stock was collated. Comparisons were made across these datasets to identify variables that affected the performance across the bridge stock.

### *4.1 Historic Seismic Demand at Bridge Sites*

In this study, the focus was on the performance of the bridge stock during earthquakes that have occurred in the last 50 years in New Zealand. The earthquakes that were assessed are summarised in Table 2, showing the distribution of event date and magnitude. The epicentre of each main event is shown in Figure 7 in Section 5, where this dataset is discussed in more detail.

**Table 2. Summary of notable damage-causing New Zealand earthquakes in the last 50 years**

| No. | Earthquake | Date | Magnitude |
|---|---|---|---|
| **1** | Inangahua Earthquake | 24 May 1968 | $M_w$ 7.2 |
| **2** | Edgecumbe Earthquake | 2 March 1987 | $M_w$ 6.5 |
| **3** | Ormond Earthquake | 10 August 1993 | $M_w$ 6.4 |
| **4** | Gisborne Earthquake | 20 December 2007 | $M_w$ 6.6 |
| **5** | Darfield Earthquake | 4 September 2010 | $M_w$ 7.0 |
| **6** | Christchurch Earthquake | 22 February 2011 | $M_w$ 6.1 |
| **7** | Cook Strait Earthquake | 21 July 2013 | $M_w$ 6.5 |
| **8** | Lake Grassmere Earthquake | 16 August 2013 | $M_w$ 6.5 |
| **9** | Eketahuna Earthquake | 20 January 2014 | $M_w$ 6.1 |
| **10** | Kaikōura Earthquake | 14 November 2016 | $M_w$ 7.8 |

The ground motion intensity at each bridge location for these ten earthquakes was defined using PGA contours from the United States Geological Survey (USGS) earthquake catalogue (United States Geological Survey, 2017). The PGA contours were defined from a combination of recorded ground motions and estimated shaking intensity from felt reports (Worden & Wald, 2016). The PGA at each bridge location, termed event PGA in this research, was approximated using the Empirical Bayesian Kriging interpolation, a geostatistical analyst tool in ArcGIS (ESRI, 2017).

An example of the PGA contours for the 2011 Christchurch earthquake in relation to the bridge locations is presented in Figure 6. This approach is not able to fully account for variation in ground motion intensity due to the soil profile at the location of each bridge, but for this high level bridge stock assessment, this approach was deemed acceptable. Most bridges are short in length (less than 50 m) with less than three spans and as such have short periods that can be approximated by PGA. Assessment using other intensity measures, such as spectral acceleration at different periods of vibration and ground motion duration, would require a level of detail, such as pier height, foundation layout, and geotechnical conditions at each bridge, that is not currently readily available in the New Zealand Highway Structures Information Management System. However, in the future, intensities could be further refined using improved regional velocity models and specific ground motion simulations for each event.

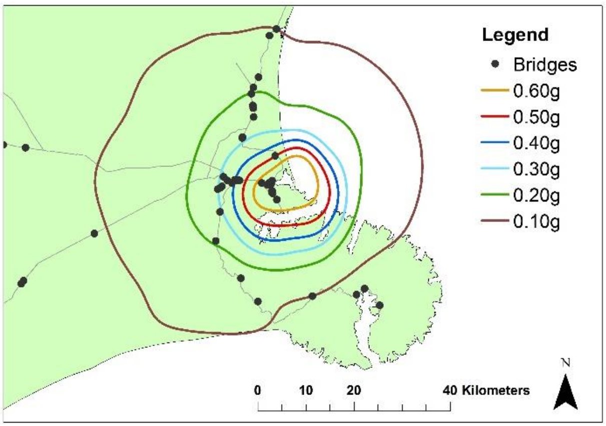


**Figure 6. PGA contours of the 2011 Christchurch earthquake and bridge locations**

### *4.2 Historic Bridge Damage Classification*

Bridge damage from historic earthquakes was defined based on details collated from post-event reconnaissance reports, commissioned reports, and journal articles (Chapman, 1993; Palermo et. al., 2010; Palermo et. al., 2011; Palermo et. al., 2017; Pender & Robertson, 1987; Shepherd et. al., 1970; Wood et. al., 2012; Wood & McHaffie, 2017). The level of detail describing the observed bridge damage from these sources varies according to the age of the earthquake, with information from recent earthquakes being more detailed than that from older earthquakes. Damage descriptions were mostly qualitative, and relate to either structural and/or geotechnical damage, with both types of damage observed at some bridges. Here structural damage refers to damage caused by the inertial response of the bridges due to earthquake excitation, while geotechnical damage refers to damage to the bridge, geotechnical structures (foundation, abutment and wingwalls), and approaches due to permanent ground deformation. Due to the differences in damage descriptions used across different reports, there were some uncertainties in the classification, therefore a qualitative approach was used. The damage severity was classified into three categories, none - minor, moderate, and major. The none-minor category was used to represent bridges that had reported a low level of damage, as well as those bridges that had no record of inspection, suggesting that any damage would not have been significant and that this damage category was appropriate. Table 3 provides some examples of how damage descriptions from different sources were translated into the severity classifications used. Where the damage descriptions for a single bridge fell into more than one category, the most severe damage category was assigned to the bridge. The damage information and damage severities were collated in a database together with characteristics of the bridges discussed previously.

**Table 3. Damage Severity and Damage Descriptions**

| Damage Severity | Damage Description | Examples of Structural Damage | Examples of Geotechnical Damage |
|---|---|---|---|
| None – Minor | Damage does not affect the structural integrity or bridge functionality | • No damage<br>• Minor cracking of structural elements<br>• Minor damage to expansion joints | • No damage<br>• Minor pavement cracking<br>• Minor soil gapping and approach fill settlement |
| Moderate | Damage results in some loss of structural integrity, and/or limited reduction of functionality (e.g. speed restrictions) | • Minor displacement of the superstructure<br>• Cracking and/or spalling at beams/piers<br>• Exposure of reinforcement at beams/piers | • Spalling, cracking, or displacement of geotechnical structures<br>• Approach fill settlement affecting bridge function |
| Major | Damage results in loss of structural integrity and/or loss of functionality | • Severe damage at piers<br>• Severe damage at beams<br>• Twisting of deck<br>• Separation of the deck from piers and abutment<br>• Noticeable displacement of structural components<br>• Superstructure shifted off bearings | • Large settlement of approaches and geotechnical structures<br>• Significant gapping or cracking of soil<br>• Significant cracking and displacement of geotechnical structures |

### *4.3 Seismic Screening and Retrofit of Bridges in New Zealand*

Seismic screening was initiated in the late 1990's to assess the seismic performance of State Highway bridges across New Zealand (Chapman et al. 2000). All the bridges in the inventory were assessed using a preliminary screening procedure to define the priorities for carrying out detailed seismic assessments and to minimize the number of structures that would require more detailed assessment. It included the elimination of bridges that did not warrant further ranking because their size or form was assumed to provide inherent resistance to significant seismic excitation levels such as culverts and single-span bridges with integral abutments. Detailed seismic assessment using non-linear pushover analyses were then carried out on those bridges that were not removed through the screening process (Novakov et al., 2017).

The preliminary screening procedure included estimating three main variables: hazard index, importance (of the bridge) index and vulnerability index. The hazard index reflected the seismicity at the bridge site and other hazards (e.g. risk of liquefaction) likely to affect the bridge structure. The PGA that may potentially result in severe damage, termed screening PGA herein, was estimated for each bridge. The estimation of this PGA was part of the risk assessment procedure, where a risk event was described based on information gathered about the seismic hazard at the bridge site and other hazards that affect the bridge structure. The screening PGA for each bridge is an estimate based on the descriptive intensity of ground shaking and general expected performance of each bridge. The assessment was dependant on the experience and judgement of the assessors without any significant analytical work, as preliminary screening was intended to define priorities for carrying out detailed seismic assessments (Chapman et al., 2000). In cases where there was more than one PGA defined for a bridge because of more than one risk event was defined, the highest PGA was used for comparison in this research. This PGA estimate is used to help compare the expected bridge response and the actual bridge response.

The main output of this screening procedure was a bridge ranking in order of priority to justify detailed seismic assessment and subsequently, assessment to determine which bridges should be retrofitted. Retrofitting was carried out on several bridges, with a large number of these involving the installation of inter-span linkages on high priority routes.

# 5 Results

## *5.1 Seismic Demand at Bridge Locations*

Figure 7 presents the locations of the bridges that experienced an event PGA of 0.05g or higher and the epicentre location for each historic earthquake assessed. These bridges are mainly distributed along the eastern to the southern part of the North Island and the northern part of the South Island, which aligns with the regions with the highest seismic hazard across the country based on the National Seismic Hazard Model (Stirling et al., 2012). An event PGA of 0.05g was exceeded 824 times across these earthquakes, with some bridges experiencing this level of shaking in multiple events.

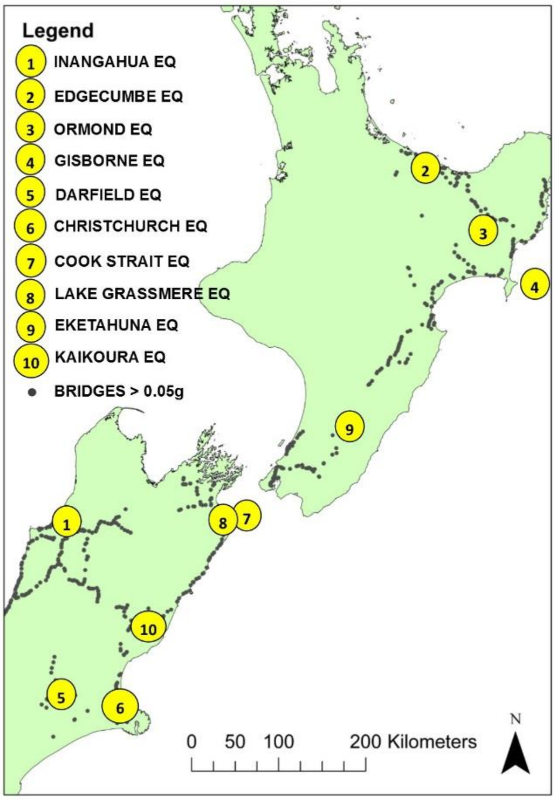


**Figure 7. Epicentres of the ten historic earthquakes and locations of bridges with event PGAs higher than 0.05g**

The distribution of the range of event PGA experienced by the bridges and the moment magnitude (Mw) of the respective earthquakes are summarised in Figure 8. The smallest range of event PGAs were experienced during the Gisborne earthquake, while the broadest range of event PGAs were experienced during the Kaikōura earthquake. The epicentre of Gisborne earthquake was offshore, about 50 km off the east coast of New Zealand's North Island, hence the intensity of ground motions affecting the bridges close to the coast was relatively small. Bridges which were affected by the other earthquakes where the epicentres were located onshore typically experienced a broader range of ground motion intensities. Most of the bridges experienced relatively small event PGA, with approximately three quarters of bridges experiencing an event PGA less than 0.30g. Figure 9 shows the distribution of bridges which experienced a PGA of 0.05g or higher based on the era of construction. Similar to the New Zealand State Highway Bridge Stock, more than half of the bridges in the collated dataset which experienced Event PGA higher than 0.05g were also constructed in Era 2, while Era 1 consists of the least amount of bridges, accounting for only about 5% of the dataset. Due to the prominence of bridges built in Era 2 in the dataset, the performance of bridges in the dataset was normalized against the respective eras to prevent any skew in the analysis of the results.

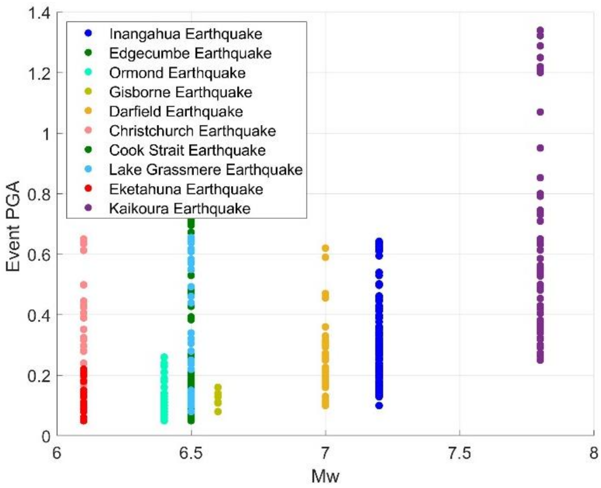


**Figure 8. Moment Magnitude and Event PGA (of 0.05g or higher) at each bridge location across all events considered**

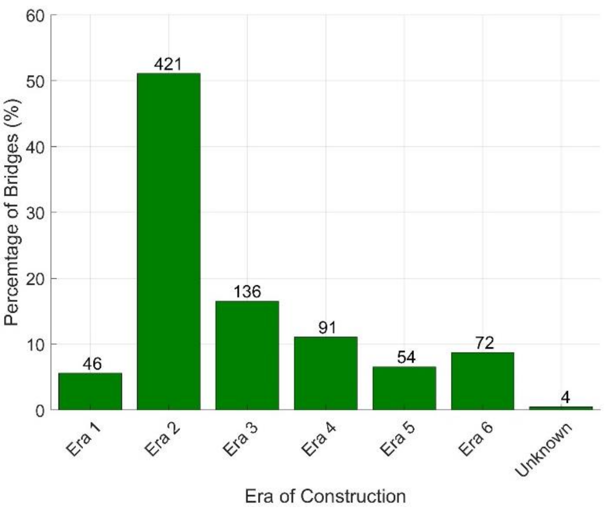


**Figure 9. Distribution of Bridges in the Dataset Which Experienced Event PGA higher than 0.05g Based on Era of Original Construction**

### *5.2 Seismic Demand and Damage Severity*

Figure 10 depicts the distribution of bridge damage severity (see Table 3) based on event PGA for both structural and geotechnical damage. For each event PGA range, data for each damage classification is presented as a ratio of the number of bridges in that classification and the total number of bridges in that PGA range. The actual number of bridges are presented at the top of each bar. As may have been expected, for structural damage, the number of bridges with none to minor damage is the highest at the lowest event PGA, between 0.05g to 0.10g. As event PGA increases, the number of bridges with none to minor damage decreases. The number of bridges with moderate and major damage is relatively small across all event PGAs as compared to bridges with none to minor damage. Overall, this data suggests that the performance of bridges in terms of structural response is generally good across all events considered. Of the many types of major structural damage observed, spalling and cracking of piers is the most common form of damage. Other commonly observed damage types were broadly confined to the superstructure, such as separation of deck from piers, translation and rotation of the superstructure, as well as damage to the piers such as residual displacement, tilt and plastic hinging.

Similar to the bridges exhibiting structural damage, the number of bridges with none to minor geotechnical damage is the highest at the lowest event PGA, between 0.05g to 0.10g. As PGA increases, the number of bridges with none to minor damage decreases. The number of bridges with moderate and major damage is relatively small across all event PGAs as compared with bridges with none to minor damage. However, as PGA increases, there is a higher ratio of bridges with moderate and major geotechnical damage, with one third of bridges experiencing a PGA above 0.80g experiencing major damage. This data again suggests generally good performance of bridges in terms of geotechnical aspects, particularly as explicit design for liquefaction was not widespread until Design Era 5 in the late 1980's. There were no observed differences in geotechnical damage based upon abutment type, with a similar number of bridges damaged for both monolithic and seat-type abutments. Of the major geotechnical damage observed, approach settlement was the most common form of damage. Other commonly observed forms of damage were damage to the abutments, (lateral displacement, tilt and plastic hinging), damage to piles (spalling, cracking, and hinging), damage to approach embankments (settlement, pavement cracking, gapping) and damage to abutment wing-walls (residual displacement and cracking). The majority of this damage resulted from liquefaction-induced lateral spreading.

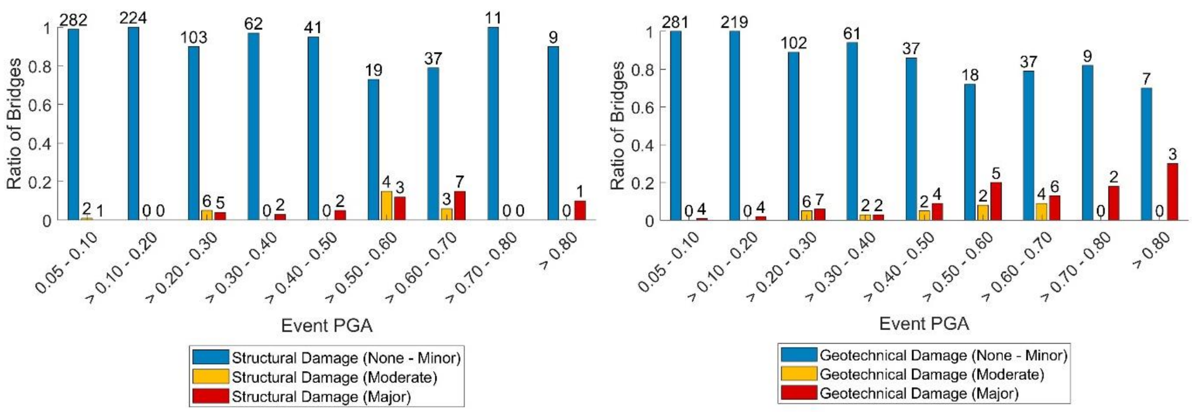


**Figure 10. Ratio of Bridge Damage Classification Based on Event PGA (a) Structural (b) Geotechnical**

Figure 11 shows the distribution of damage severity based on event PGA and the era of original construction. In this figure the number of bridges experiencing a particular event PGA band for each era is presented as a percentage of the total number of bridges in each construction era that experienced an event PGA greater than 0.05g. This normalization is to control for any skew in the presentation of the data as a result of the significant number of bridges in the dataset that were constructed in Era 2. The actual number of bridges experiencing a particular event PGA band is presented above each bar in the figure. The results are as expected for none to minor damage, with a larger percentage of bridge at lower event PGA levels across all eras, tapering off to fewer bridges at the higher event PGA levels. There are no clear observed differences in these trends across the different eras despite the varied design and construction practices. For moderate and major damage, there is evidence of damage to bridges in Era 2 and Era 3 at relatively low event PGA levels (<0.2g), and no evidence of this in other eras. However, the total count of these cases is also low, and as such these may not be entirely representative. Retrofit details across the bridge stock were collated as part of this research, however these were not discussed further here as they were shown to not play a significant role in the performance of bridges in historic events (as few were exposed to significant levels of shaking post-retrofit).

Figure 12 shows the distribution of damage severity based on event PGA and bridge length. Here, the number of bridges experiencing a particular PGA band for each bridge length band is presented as a percentage of the total number of bridges in each bridge length band that experienced an event PGA greater than 0.05g, in order to normalize the large number of bridges shorter than 50 m. The actual number of bridges in each category are presented above each bar in the figure. Most of the bridges with none to minor damage have lengths less than 50 m (65%). In the remainder of the data set, 21% of bridges have lengths between 50 – 100 m, 13% between 100 – 500 m, and 1% above 500 m. For all categories, most bridges experienced low event PGA levels, tapering off to fewer bridges at the higher PGA levels. The number of bridges with moderate to major damage is comparable for bridge lengths of less than 50 m and 50 -100 m. There are slightly more bridges with moderate and major damage with bridge length of 100 – 500 m, but the number of bridges in these cases is low, and is not as significant when compared to the number of bridges in each band. As depicted in Figure 3, the number of bridges with length above 500 m is relatively low when compared to other lengths. The bridges in this category only experienced event PGA of less than 0.4g, and none of them exhibited moderate or major damage. Although nothing definitive can be taken from these trends, there is possibly some suggestion that length affected performance. This possibility is discussed in more detail in subsequent sections.

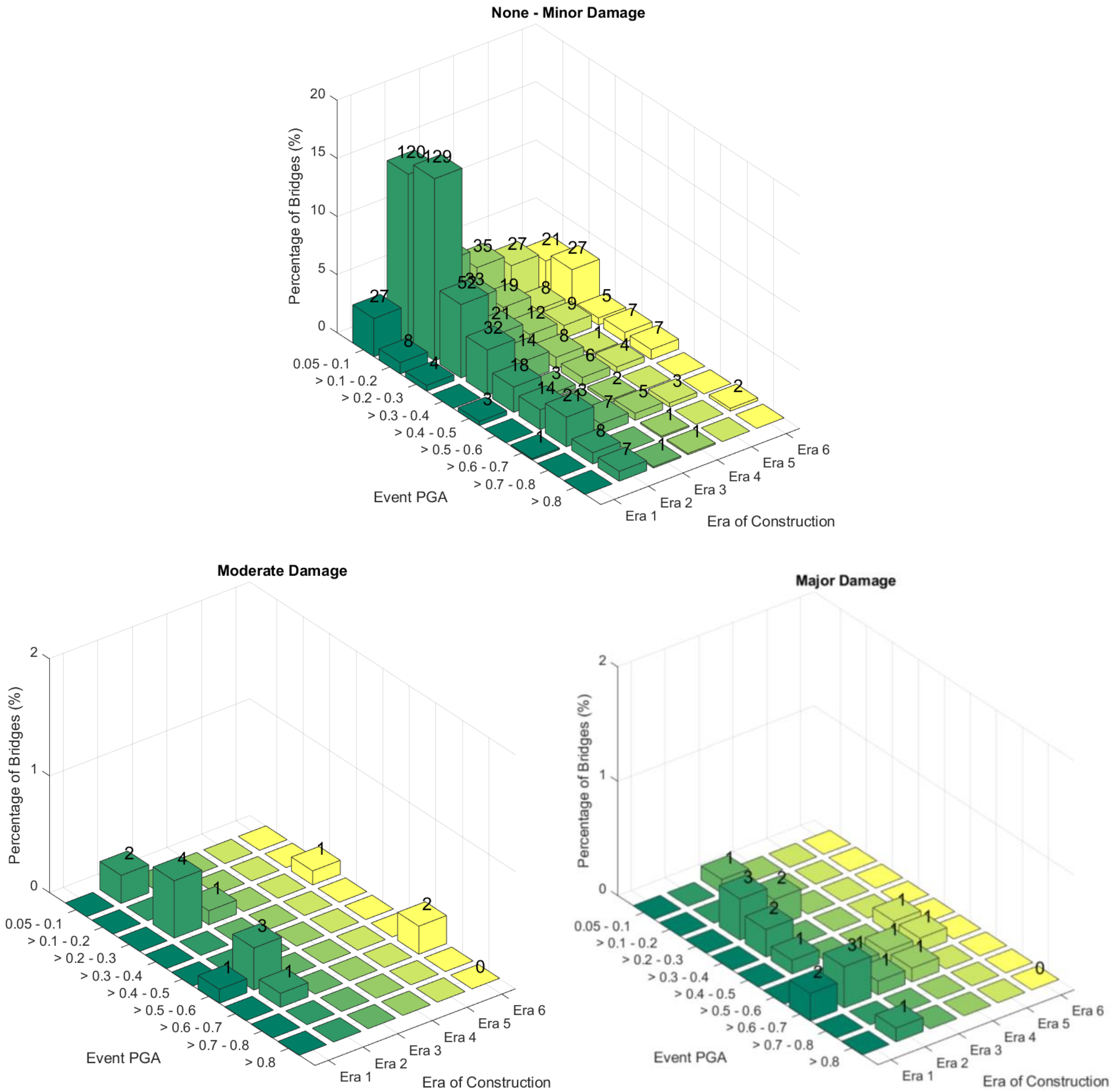


**Figure 11. Distribution of Structural Damage for Full Dataset Based on Event PGA (of 0.05g or higher) and Era of Original Construction**

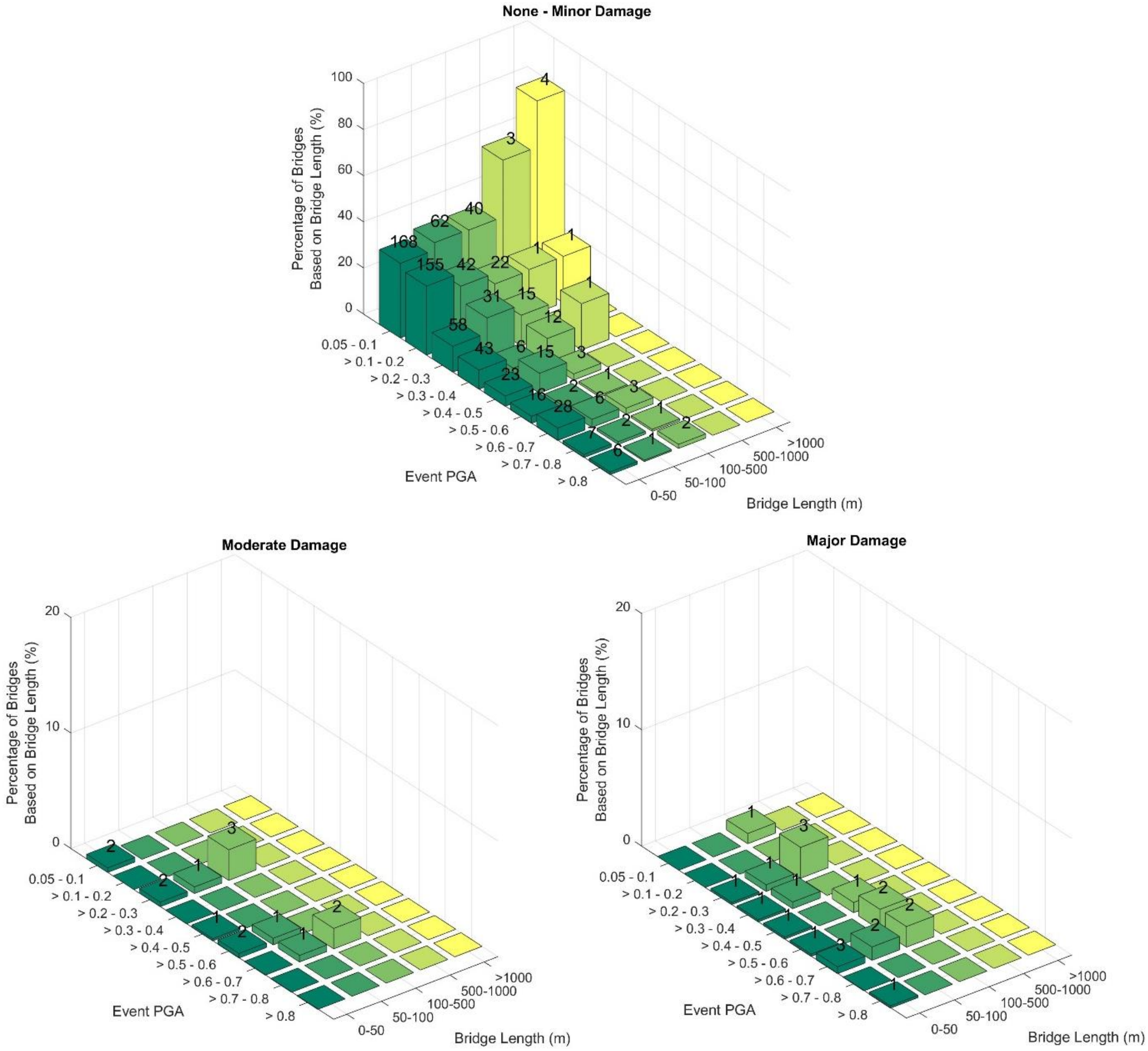


**Figure 12. Distribution of Structural Damage for Full Dataset Based on Event PGA (of 0.05g or higher) & Bridge Length**

A low percentage of bridges built in Era 6 experienced major damage across all events. While the use of ductile detailing standards would have helped with post-yield performance of Era 6 bridges, it should also be noted that Era 6 bridges were designed using a much higher return period than previous eras, resulting in significant strength and stiffness increase. In areas of medium seismicity such as Napier, bridges with short periods (0.5 s) built before 1987 were designed for about one third to two thirds the base shear of Era 6 (Hogan et al., 2013). While it is likely that the larger design base shear would have contributed to the lower incidence of damage observed in Era 6 bridges, the limited number of Era 6 bridges exposed to strong shaking was limited, and a larger dataset would be needed to confirm performance characteristics.

The case history data has been compared with examples of fragility functions developed in other international studies for bridge characteristics that could be representative of some portion of the New Zealand bridge stock. It includes 2 to 4-span bridges with concrete or steel superstructures and either single column piers, multi-column piers or wall piers. The level of detail of the characteristics of the bridge varies across different references. Figure 13a summarises the fragility functions for slight damage and Figure 13b summarises the number of bridges with none to minor structural damage based on event PGA from the New Zealand case history data. The fragility curves reach a probability of 1.0 at approximately 0.30g, however there are a number of bridges that experienced an event PGA higher than 0.30g in the New Zealand dataset, which would have exceeded the performance of some of the fragility curves. This comparison demonstrates that the performance of New Zealand bridges are not well captured by these studies, and suggest that the development of fragility curves for New Zealand typology bridges would be useful.

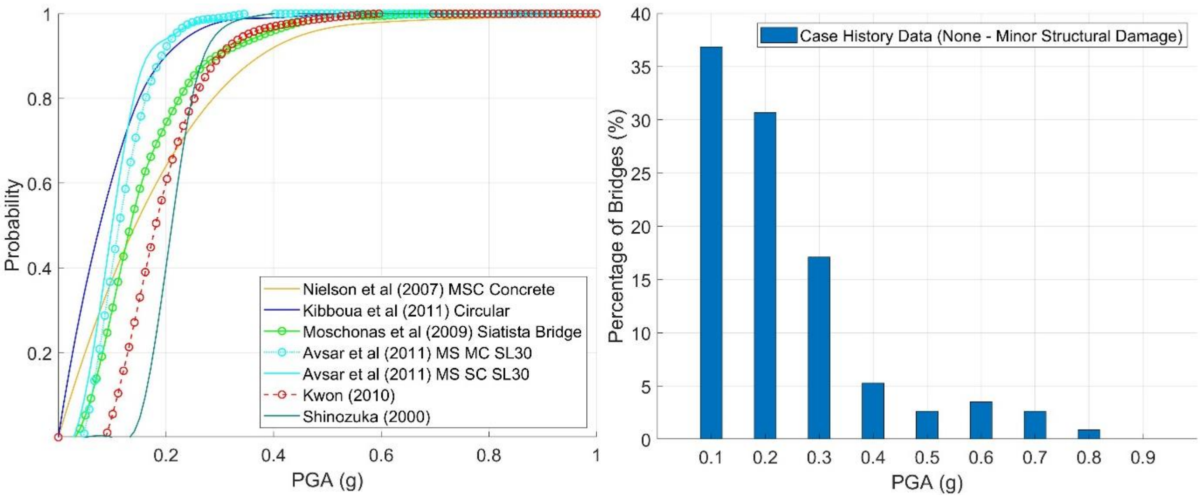


**Figure 13. (a) Internationally Developed Fragility Functions for Bridges (Slight Damage) (b) Distribution of Case History Data (None – Minor Structural Damage)**

### *5.3 Comparison with Seismic Screening*

To further interrogate the historic performance summarised in the previous section, the event PGA and screening PGA characteristics were compared. Only structural damage is discussed, as it was the main focus of the screening process. Of the 824 bridges experiencing an event PGA higher than 0.05g, 284 had a screening PGA assigned as part of the seismic screening process. Figure 14 summarises the comparison of event PGA of the full dataset and the event PGA experienced by the 284 bridges assessed, which we refer to as the screening dataset. 89

bridges experienced an event PGA between 0.05g to 0.10g, accounting for 36% of the total number of bridges assessed. Most of the bridges experienced low to moderate demands with, 77% of bridges experiencing an event PGA below 0.30g. This distribution is comparable and representative of the distribution of the full dataset, where three quarters of bridges experienced an event PGA of less than 0.30g. The distribution in terms of the era of construction and bridge length is also similar to that shown in Figure 11 and Figure 12.

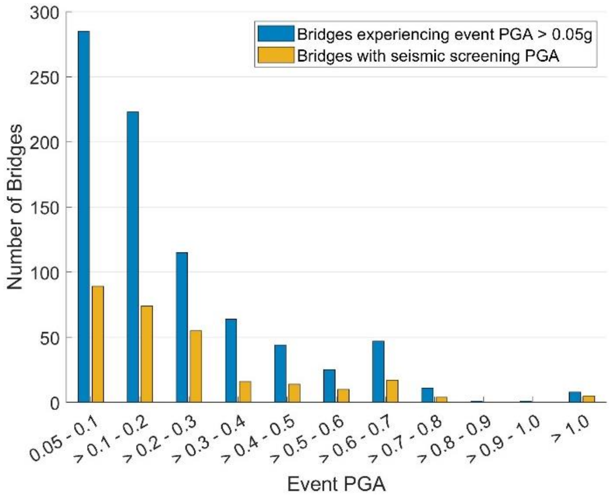


**Figure 14. Comparison of Event PGA of the Bridges in the Dataset Which Experienced Event PGA higher than 0.05g and Bridges in the Dataset with Screening PGA**

The relationship between the event PGA and the screening PGA experienced by the 284 bridges assessed is summarized in Figure 15(a). As expected, most of the bridges experienced an event PGA less than the screening PGA (85%) due to the screening PGA being related to major damage of the bridge. Some bridges experienced relatively high event PGA as compared to the screening PGA, with a number of these cases occurring during the 2016 Kaikōura earthquake. Figure 15(b) shows the distribution of bridges with structural damage based on PGA ratio, where PGA ratio refers to the ratio of the event PGA to the screening PGA. The PGA ratio is divided into three categories to differentiate between cases where damage may or may not have been expected. A PGA ratio less than 0.75 suggests that significant damage was not expected, while a PGA ratio greater than 1.25 suggests that damage may have been expected. A PGA ratio between 0.75 and 1.25 suggests that damage may be possible. Based on the findings, 90% of the bridges assessed have either no or minor structural damage, and only 10% of the bridges have moderate or major damage. Most of the bridges which suffered either no or minor damage experienced a PGA ratio of less than 0.75. As expected, the higher the PGA ratio, the smaller the number of bridges with either no or minor structural damage. Bridges with moderate and major damage are distributed quite evenly across the different PGA ratios.

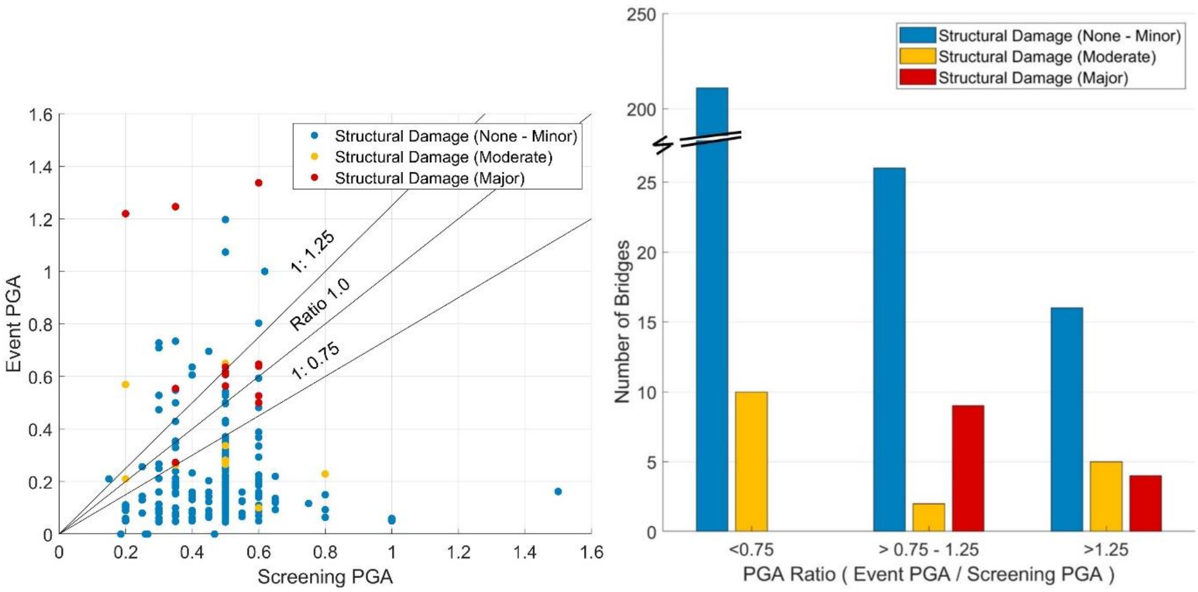


**Figure 15. (a) Distribution of the Data Based on Screening PGA & Event PGA; (b) Distribution of Data Based on PGA Ratio**

Bridges with a PGA ratio of less than 0.75 that developed structural damage, and the bridges with a PGA ratio of more than 1.25 that developed none to minor structural damage are of particular interest. The former category could suggest that the bridge performance was worse than expected, while the latter could suggest that performance was better than expected. Of the bridges with a PGA ratio of less than 0.75, ten bridges developed moderate structural damage while none developed major structural damage. The moderate levels of damage resulted in no significant loss of functionality for these cases, and as a result the performance was likely still comparable to the seismic screening assessments. As such, there is little evidence of systematic poor performance of any bridge typologies.

Sixteen bridges experienced a PGA ratio of more than 1.25 that developed none to minor damage. All had either precast pre-tensioned concrete or cast-in-situ reinforced concrete superstructures. Shorter bridges in this grouping, with lengths less than 10 m, may have experienced either no or minor structural damage as abutment response can introduce significant stiffness and energy dissipation and the bridge can behave as a locked "locked-in" structure, which moves in-phase with the surrounding ground. These bridges all have wall type piers that likely have higher capacity than was originally assessed. Longer bridges in this group could have also been strongly influenced by travelling ground wave effects that result in a phase lag between the seismic input motions at the piers along the length (Wood et al., 2012). This could be linked to the fact that some bridges which have been overdesigned in the 1940s to 1950s (Hogan et al., 2013; Palermo et al., 2010), and the likelihood of seismic capacity for the bridges built in 1930s to exceed design levels due to structural configurations (Hogan et al., 2013).

## 6 CONCLUSIONS

This study has assessed the historic seismic bridge performance of the New Zealand highway bridge stock from the 1968 Inangahua earthquake through to the 2016 Kaikōura earthquake. From the geospatial analysis conducted, there were over 800 instances of bridges experiencing a PGA higher than 0.05g, with some bridges experiencing this level of shaking in multiple earthquakes. These bridges are mostly distributed along the eastern to the southern part of North Island and the northern part of South Island, aligning with regions with the highest seismic hazard across the country. Most of the bridges experienced small to moderate PGA levels, with approximately three quarters of the bridges experiencing PGA below 0.30g.

The number of bridges with moderate and major damage is relatively small, only 4% across all event PGA as compared with bridges with none to minor damage. Overall, this data suggests that the performance of bridges in terms of structural response is generally good across all PGA levels. Similar to the bridges exhibiting structural damage, the percentage of bridges with moderate or major geotechnical damage is relatively small, only 2% and 4% respectively across all PGA levels. Of the many types of structural damage observed, spalling and cracking of piers was the most common form of damage. Of the major geotechnical damage observed, approach settlement was the most common form, resulting from liquefaction-induced lateral spreading. There was no clear difference in geotechnical damage based on abutment type, with a similar number of bridges damaged with monolithic and seat-type abutments. Although most of the bridges which experienced none to minor structural damage were constructed in Era 2 (50%) and have bridge lengths between 0 m to 50 m (65%), based on the comparison of bridge performance across different eras and bridge lengths, no clear differences were observed despite the varied design and construction practices. The results are as expected for none to minor damage, with a larger percentage of bridges at lower event PGA levels across all eras.

Comparison of the data with the NZ Transport Agency seismic screening results suggests that the performance of bridges was generally good. Of the bridges that experienced an Event PGA smaller than Screening PGA, none developed major structural damage, and only

10 developed moderate structural damage with no significant loss of functionality. Of the bridges that experienced a larger Event PGA than Screening PGA, 16 developed none to minor damage. Some shorter bridges may have performed better than expected due to the effects of abutment damping and stiffness. Longer bridges might have performed better due to travelling wave effects that results in a phase lag between the seismic input motions at the piers along the length. Other factors which were not considered in this research, such as site conditions, geometry and orientation of the bridge could also have influenced performance. These factors could be accounted for in future specific assessment of these case histories, together with the use of other intensity measures for more site-specific assessment of bridge case histories.

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