DEVELOPMENT OF NEW ZEALAND SEISMIC BRIDGE STANDARDS

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SUMMARY

During seismic assessments of bridges where there is a lack of construction documentation, one method of determining likely structural detailing is to use historic design standards. An overview of the New Zealand bridge seismic standards and the agencies that have historically controlled bridge design and construction is presented. Standards are grouped into design era based upon similar design and loading characteristics. Major changes in base shear demand, ductility, foundation design, and linkage systems are discussed for each design era, and loadings and detailing requirements from different eras were compared to current design practices. Bridges constructed using early seismic standards were designed to a significantly lower base shear than is currently used but the majority of these bridges are unlikely to collapse due to their geometry and a preference for monolithic construction. Bridges constructed after the late 1970s are expected to perform well if subjected to ground shaking, but unless bridges were constructed recently their performance when subjected to liquefaction and liquefaction-induced lateral spreading is expected to be poor.

INTRODUCTION

Seismic screening programs have successfully been implemented in New Zealand to identify potentially vulnerable bridges and to determine which bridges represent priorities for detailed assessment and retrofit [1]. As the evaluation of seismically vulnerable bridges shifts focus from a rapid screening towards the assessment of individual bridges, one of the major challenges in performing these detailed assessments is the lack of construction documentation on which to base member strength and detailing. This problem is particularly acute for bridges owned by local authorities due to documentation being lost when small authorities amalgamate together or never being of high standard when originally prepared.

One method to assess bridges having limited documentation is to use design standards and bridges of similar form from the same design era to determine likely material properties and detailing, but this information is currently widespread and difficult to locate, making it an inefficient method of assessment. In response to the need to compile this information, historic seismic bridge design practices in New Zealand are summarized. This summary includes an overview of the organizations that have historically controlled New Zealand bridge design requirements and a review of the design requirements for various design eras. Base shear, ductility, foundation, and linkage system requirements of each design era are compared to current design practices to provide guidance on likely seismic behaviour of bridges built in previous design eras with respect to current design practices.

ORGANIZATIONS CONTROLLING BRIDGE DESIGN AND CONSTRUCTION

Throughout much of the 19th and 20th centuries, bridge design and construction on the New Zealand State Highway network was managed by a central government agency, the earliest being the Public Works Department (PWD). Established by the Immigration and Public Works Act of 1870, the PWD provided oversight for all government works projects, but a series of District Road Boards controlled the surveying, building, and maintenance of roads [2]. Control of road works passed to the Survey Department in 1889 until the Department of Roads took over construction following its establishment in 1901. Transfer of road control finally returned to the PWD when the Department of Roads was absorbed back into the PWD in 1908 [3].

The Main Highways Act of 1922 created the Main Highways Board, composed of PWD officers and officials appointed by the New Zealand Governor General and the Minister of Works [4]. While the PWD continued to operate and oversee design and construction projects, authority over approval and financing of road and bridge projects was transferred to the Main Highways Board. The Main Highways Board began operating in 1924 and within its first year, declared over 9,600 km of roads as main highways, forming the basis of the current State Highway network. In 1936 a number of these main highways were officially renamed State Highway and the responsibility for improvements and maintenance was placed with the Main Highways Board and its District Offices [5].

The Ministry of Works Act [6] established the Ministry of Works (MoW) to replace the PWD and take over the portfolio

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of work previously held by the PWD. While the MoW had officially replaced the PWD, the PWD was occasionally referred to in documents published after this transition. In 1953 the National Road Act was passed, replacing the Main Highways Board with the National Roads Board in 1954. In 1959 a separate Roading Division of the MoW was created to supervise the vast amount of maintenance, construction and management involved with the State Highway network [3].

The Public Works Act of 1928 was amended in 1973 and the MoW was renamed the Ministry of Works and Development (MWD) which operated until construction and asset management activities were privatized in 1988 with the passing of the Ministry of Works and Development Abolition Act. The Transit New Zealand Act of 1989 abolished the National Roads Board and gave control of construction, maintenance, and planning of the State Highway network to Transit New Zealand (TNZ), a Crown agency in the Ministry of Transport, while road and bridge projects were financed through the Land Transport Fund. The design, construction, and research arms of the former MWD were transferred to the government-owned Works and Development Services Corporation, which was forced to compete with private companies for public infrastructure work [7]. The Works and Development Services Corporation was sold in 1996 and has operated as Opus International Consultants since 1997 [8]. The crown entity Transfund New Zealand was created in 1997 under the Transit New Zealand Amendment Act No. 2 1995 to divide government funding between Transit New Zealand and regional authorities. The Land Transport Management Act of 2003 merged Transfund New Zealand with the Land Transport Safety Authority in 2004 to form Land Transport New Zealand (LTNZ). LTNZ merged with TNZ after the passing of the Land Transport Management Act Amendment of 2008 to form the New Zealand Transport Agency (NZTA). NZTA currently manages operation and funding of the State Highway network.

DEVELOPMENT OF SEISMIC BRIDGE STANDARDS

Seismic bridge design has been controlled by several standards published by NZTA and its preceding organizations. These standards defined the requirements for traffic, wind, flood, temperature and seismic loading and either contained requirements for member design, and detailing of various materials or referenced the appropriate material standard developed for the building industry. Additionally, MWD released several supplementary design briefs on various aspects of bridge design in an effort to disseminate best practice and research available at the time of publication.

An overview of the major changes in seismic design requirements for New Zealand bridges is provided in the following sections. The requirements discussed are those that apply to bridges that can be analysed using an equivalent linear static analysis (ESA) approach. Changes in bridge analysis using modal or time history methods are not discussed here because most New Zealand bridges meet the requirements for ESA and are unlikely to have been designed using an alternative method. The discussion of design standard changes is organized into two different aspects of seismic design: seismic loading, and member detailing requirements. Changes in seismic loading refer to changes in base shear computation, design spectra and seismic hazard zones. The development of detailing requirements focuses on foundation design, inter-span linkages and seat lengths at supports. A discussion regarding the reinforcement detailing of concrete piers is also outlined to highlight major shifts in bridge design philosophy, but this discussion is intentionally kept brief as the historical changes in seismic detailing of reinforced concrete have been thoroughly described by Fenwick and MacRae [9].

The bridge standards outlined in the following sections are organized based upon similar design requirements and philosophies into the following design eras:

- Era 1 (pre 1930s): No Seismic Standards.
- Era 2 (1930s to mid-1960s): Early Seismic Standards and Elastic Design.
- Era 3 (mid-1960s to mid-1970s): Preliminary Ductile Standards.
- Era 4 (mid-1970s to late 1980s): Early Ductile Standards.
- Era 5 (late-1980s to early-2000s): Basis of Current Standards.
- Era 6 (early-2000s to Present): Current Standards.

Boundaries between these eras are not clearly defined and bridges designed close to these boundary years may contain characteristics of either the preceding or following design era. Seismic loading and design requirements for each design era are summarized in the following sections.

Era 1 (pre 1930s): No Seismic Standards

No seismic provisions appear in New Zealand bridge standards published prior to 1931 when, in response to the 1931 Hawke’s Bay earthquake, the Draft General Earthquake Building By-Law was presented to the New Zealand House of Representatives. While several major earthquakes occurred prior to 1931, and some bridge designers may have made some considerations for seismic design, bridges built before 1931 are assumed to have been designed without the application of seismic loading [10]. Concrete bridges of this era are likely to have integral abutments and superstructures cast monolithically with piers due to the preference for constructing bridges using cast-in-situ concrete.

Era 2 (1930s to mid-1960s): Early Seismic Standards and Elastic Design

The first seismic provisions for bridge design were introduced in 1933 within the Public Works Department Road Bridges, Loads and Allowable Stresses (RB&LAS) in response to the 1931 Hawke’s Bay earthquake. Prior to this standard the governing horizontal loading for bridges was flood loading [11]. Seismic standards were updated eleven years later with the release of the 1944 Highway Bridge Design: Tentative Preliminary Code (HBD-TPC) and again in 1956 when the New Zealand MoW published the first Bridge Manual. The Bridge Manual, modelled after the AASHO Standard Specifications for Highway Bridges [12], was intended to describe the existing best practice on bridge design and construction. The manual provided requirements for superstructure and substructure component design and detailing for a range of materials and soils.

All of the Era 2 standards required bridges to resist the same base shear irrespective of bridge geometry or location. Members were designed with working stress design methods in which the stress in the member was to be kept below an allowable stress defined for a given failure mode (e.g. a percentage of yielding stress of the reinforcement or crushing of concrete). Stiffness of reinforced concrete members was determined from gross section properties.

Era 2 Seismic Loading

All of the Era 2 seismic standards required bridge piers to be designed to resist a lateral force equal to 0.1g x the mass of the superstructure, and this force was distributed to the piers based upon tributary area. The 1944 HBD-TPC introduced an increase in the allowable stress for earthquake loading to 133% of the normal working stress. The 1956 Bridge Manual maintained this increase in working stress but required bridges to resist this 0.1g load applied as a continuous horizontal force at the centre of mass of the structure [13]. It is unclear whether the 1933 RB&LAS or the 1944 HBD-TPC applied the horizontal force in this manner.

Era 2 Seismic Detailing

The 1933 RB&LAS contained very few requirements for seismic detailing, and the 1944 HBD-TPC had none at all. Although the 1933 RB&LAS did not prescribe any specific detailing requirements, it did require that all concrete include a small percentage of reinforcement even if not required by direct loading. No guidelines were provided to determine this minimum reinforcement ratio and therefore the amount of reinforcement was left to the designer's discretion. The 1933 RB&LAS encouraged that where possible bridges should have superstructures monolithic with piers and abutments, or if monolithic bridges were impractical, then bridge components were to be well tied together. No guidance was provided for the design of linkage systems to tie the bridge together.

The 1956 Bridge Manual provided some of the first seismic detailing requirements, such as keeping longitudinal reinforcement splices out of areas of peak stress arising due to lateral forces. Guidance was provided for designing pile foundations to resist earthquake forces in flexure, but typically this wasn't practiced until the 1960s as prior to this time lateral load was assumed to be resisted only by raked piles [14]. If piles were used to resist earthquake forces in stiff clays or dense gravels, the point of fixity was assumed to be 3 m below surface. Abutments were designed to only resist seismic forces arising from their self-weight and supported bridge spans. The seismic force from the approach fill was ignored.

In 1956 an instructional document was released to supplement the Bridge Manual in the design of inter-span linkage systems. While a strong preference towards monolithic bridges was still held by the MoW, the document was released to provided consistency of design for linkage and hold-down systems required for the numerous prefabricated bridges being built at the time [15]. Designers were encouraged to use flexible rather than rigid piers for bridges that required linkages in order to reduce the demand on the linkages. Examples of standard linkage systems were provided for steel girder and precast concrete construction.

In 1957 the MoW issued a set of Standard Plans for Highway Bridges to economize bridge design and construction [16]. The plans included details for steel truss, steel girder, reinforced concrete slab, and reinforced concrete "T" beam superstructures of varying lengths. Details were provided for reinforced concrete pier walls to be used for three span bridges of prescribed length and height, but no standardized details were provided for piers of other forms (e.g. multi-column piers). Standard designs for reinforced concrete piles were also incorporated for both square or octagonal cross sections either 14" (356 mm) or 16" (406 mm) in width.

Era 3 (mid-1960s to mid-1970s): Preliminary Ductile Standards

The seismic loading standard used for buildings was updated in 1965 with the publication of NZS 1900 Chapter 8:1965 [17]. While there was no update of the 1956 Bridge Manual to include the provisions of this new standard, it was common practice to adopt the provisions in NZS 1900:1965 for bridge design [18]. The 1956 Bridge Manual was superseded in 1971 when the MoW published the first of a series of Highway Bridge Design Briefs (HBDB) [19]. The next revision of the HBDB was issued in November 1972 [20], and reissued in July 1973 to update the 1972 HBDB with metric units and to provide better guidance on the design of inter-span linkages and seismic loading of earth retaining structures [21].

The approval of these standards discontinued the practice of applying a 0.1g horizontal force to represent the seismic load on a bridge, regardless of geographic location or structural characteristics, by introducing both seismic zones and period dependent seismic coefficients. The use of ductility as a means of limiting seismic actions on the bridge was introduced in these standards. For each revision of the HBDB, member design was based on ultimate strength methods, while foundations and earth-retaining structures continued to be designed based upon working stress methods.

Era 3 Seismic Loading

In NZS 1900:1965 New Zealand was divided into three seismic zones: Zone A, Zone B and Zone C (Figure 1). Two different sets of design spectra were used depending upon whether the building was either publically or privately owned. The spectra for public buildings were adopted to determine the seismic coefficient for bridge design (Figure 2), and the base shear was calculated by simply multiplying the seismic coefficient by the weight of the bridge. The Zone A design spectrum was based on a smoothed elastic response spectrum of magnitude similar to that obtained from the largest horizontal component of the 1940 El Centro earthquake [22]. Zone A was linearly scaled by 75% and 50% to obtain the Zone B and Zone C spectra, respectively (Figure 2). The maximum seismic coefficient from these spectra was 0.16g for bridges with a fundamental period less than 0.44 s and located in Zone A. These spectra and seismic zones were used for bridge seismic design until the mid-1980s.

Seismic provisions in NZSS 1900:1965 were adopted in the 1971 HBDB with bridges assumed to be designed with a global ductility factor of four. In the 1971 HBDB it was stated that in order to achieve this ductility factor, the local ductility at the location of plastic hinges must be much larger than four. Guidance on what detailing was required to achieve this level of ductility was provided in Appendix B, which was a reproduction of the guidelines in the 1970 Code of Practice for the Design of Public Buildings [23].

In the 1972-73 HBDB the loadings provisions of the 1971 HBDB were expanded to classify bridges into two categories of seismic response: i) ductile structures and ii) partially ductile or non-ductile structures. In the 1972-73 HBDB, it was recognized that while it was preferable to design ductile structures that resisted seismic loads by providing plastic hinges in predictable and accessible locations, bridge geometry or economic considerations may make it impractical to achieve the required ductility factor. Both types of structures required seismic design to meet the performance criteria of collapse prevention and the ability to service light traffic post-earthquake.

The base shear calculation in the 1972-73 HBDB for ductile structures was identical to the 1971 HBDB except that an importance factor was introduced to reduce the base shear for bridges that were less critical to the State Highway network (Equation 1). The importance factor (F) in Equation 1 ranged from 0.7 to 1.0 and was based upon the average daily traffic volumes that the bridge serviced, as outlined in Table 1. The total base seismic base shear (V) in the direction being considered was calculated as follows:

$$V = CFW \tag{1}$$

Where:

C = Basic seismic coefficient

F = Importance factor

W = Total load subject to seismic acceleration

The basic seismic coefficient was determined for the appropriate seismic zone and fundamental period using the design spectra in Figure 2. The base shear calculated using Equation 1 increased the overall ductility factor from four, used in the 1971 HBDB, to six and assumed considerable post-elastic energy absorption in the bridge. While the target global ductility increased, there was no change in base shear because there was an incomplete knowledge about what detailing was required to achieve this level of ductility. Instead of raising the required base shear, the MoW encouraged designers to make the bridge as ductile as possible.

Table 1. 1972-73 HBDB Importance Factor

Category	Min. Value of (F)
Bridges carrying 2,500+ vehicles per day; all bridges under or over motorways or railways	1.0
Bridges carrying 250-2,500 vehicles per day	0.85
Bridges carrying less than 250 vehicles per day	0.70

For bridges whose structure of member geometry provided inherent strength that exceeded the effects of the maximum elastic response, a separate loading criterion was proposed. Base shear was still calculated using Equation 1, but the combined values of the base shear coefficient and importance factor (CF) were defined in Table 2.

Era 3 Seismic Detailing

In the 1971 HBDB some preliminary guidelines on capacity based design were introduced by requiring that all elements have sufficient strength to transmit forces to the plastic hinges. Damage was to be limited to plastic hinge zones and away from brittle elements. A preference was expressed for resisting seismic loads by flexure in the piers rather than by direct connection to one rigid element such as an abutment. To ensure that ductility was concentrated in the piers and away from the superstructure, the 1971 HBDB required that the sum of all superstructure elements connecting to the pier have an ultimate moment capacity 15% greater than the top of the pier. Moment capacities of resisting members were calculated using the strength reduction factor $\phi = 1$ (i.e. using expected member capacities rather than dependable strengths).

The 1972-1973 HBDB revisions expanded the capacity based design guidelines introduced in the 1971 HBDB and clarified

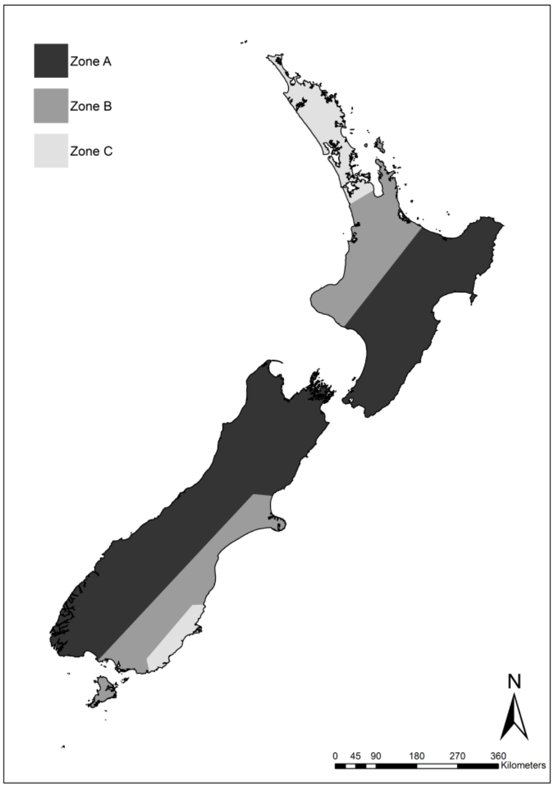

Figure 1:* *Seismic zones from NZS 1900: Chapter 8. Used from 1965 – 1987.

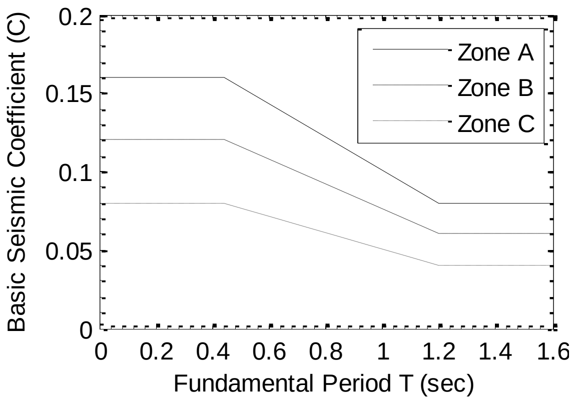

Figure 2:* *Basic seismic coefficient for public buildings from NZS 1900: Chapter 8. Used in seismic design of bridges from 1965 to 1987.

Table 2. 1972-73 HBDB values of CF for use when lower standard of earthquake resistance is chosen due to economic reasons.

Category	Zone A	Zone B	Zone C
1	0.24	0.18	0.12
2	0.20	0.15	0.10
3	0.17	0.13	0.09

the calculation of plastic hinge moments. When determining the likely plastic hinge moment, an overstrength factor of 1.25 was applied to the yield stress of the reinforcing steel (f_y) of the plastic member to account for strain hardening effects. Ultimate moment capacities of members resisting this plastic hinge moment were calculated based upon a reinforcing steel yield stress of $1.15f_y$, but no increase of yield stress in shear reinforcement was allowable in order to avoid brittle shear failures. Additionally, only mild steel (HY40/Grade 275, f_y = 40 ksi or 275 MPa) was allowed in areas of plastic hinging. High strength, low ductility reinforcing steel such as HY60 steel (Grade 380, f_y = 60 ksi or 380 MPa) was not allowed within plastic hinge zones but could be used in members resisting the plastic hinge moment.

Abutment backwall design requirements were updated from the 1956 Bridge Manual to include provisions for the wall moving either towards or away from the approach fill. In both cases inertial forces from the superstructure that were transferred through either bearings or tie-backs were applied to the abutment, but inertial forces from the self-weight of the abutment were ignored. The earth pressures assumed to act on the abutment when it was moving away from the wall were the combined active and earthquake earth pressures, and the at-rest (static) earth pressure was used when the wall was moving towards the approach fill. Earth pressures were determined using Coulomb wedge theory and design methods were described in CDP 702/C: Retaining Wall Design Notes [24].

Linkage design requirements first appeared in the 1972-73 HBDB. Linkages between spans were designed to resist 20% of the inertial load from the heavier of the two adjacent spans. No explicit guidelines were provided for sizing support lengths to avoid span unseating, suggesting that the 1972-73 HBDB assumed that linkage bolts would be adequately sized to prevent unseating.

Along with the revisions in Era 3 seismic detailing, the 1957 Standard Plans for Highway Bridges were reissued in 1970. This issue included details for a variety of precast concrete superstructure beam types and post-tensioned concrete "I" sections. Octagonal prestressed concrete piles were included in the same 14" and 16" sizes as their existing reinforced concrete counterparts.

Era 4 (mid-1970s to late 1980s): Early Ductile Standards

The 1972-73 HBDB was amended in 1976 and reissued in 1978 [25]. These amendments were similar to previous versions of the HBDB except that during this era there was a widespread use and understanding of capacity-based design principles, with the 1976-78 HBDB providing guidelines on how to detail bridge piers to achieve a desired ductility.

Era 4 Seismic Loading

The calculation of basic seismic coefficient in the 1976-78 HBDB remained unchanged from the previous versions. The seismic loadings code NZS 4203:1976 [26] was referenced to define seismic zonation, but no update was made to include the new spectra for flexible soil sites or the increase in spectral accelerations for periods over 0.44 s that were defined in NZS 4203:1976. However, in the 1976-78 HBDB the role of foundation rigidity on assumed loading was acknowledged. Rigid foundations on firm ground were assumed to provide 5% structural damping, and bridges with such foundation conditions were required to be capable of reaching a global ductility of six. The ductility demand was reduced for bridges founded on flexible soils based on the assumption that these soils would provide additional damping. Criteria for flexible soil sites were given in NZS 4203: 1976 and are summarized in Table 3.

Table 3. NZS 4203:1976, 1984 Flexible Soil Criteria

Soil Type and Description		Depth of soil (m)
Cohesive Soil	**Average undrained shear strengths (kPa)**	
	50	6
	100	8.5
	200	12
Cohesionless Soil		
Sands		15
Gravels		15

Era 4 Seismic Detailing

The 1976-78 HBDB was the first bridge standard to explicitly provide guidance for detailing bridge piers to achieve a given ductility by referencing Ministry of Works CDP 810/A: Ductility of Bridges with Reinforced Concrete Piers [27]. In addition to the detailing guidelines in CDP 810/A, the 1976-78 HBDB prohibited the use of Grade 380 reinforcing steel in plastic hinge zones but allowed the use of Grade 380 in members resisting the plastic hinge moment. Otherwise, the 1976-78 HBDB remained unchanged from previous versions.

In 1978 the MWD replaced the previous Standard Plans for Highway Bridges with CDP 901: Standard Plans for Highway Bridge Components, also known as the Blue Book [28]. The Blue Book provided schematic layouts and likely plastic hinge locations for the piers of single span, two span, three span and multi-span bridges. All superstructure components were updated to conform to HN-HO-72 loading and standard components for superstructure linkages systems were included. Pile sections were also updated for HN-HO-72 loading, with axial force-bending moment interaction diagrams and design charts included along with the reinforcement detailing.

Era 5 (late-1980s to early-2000s): Basis of Current Standards

The HBDB was updated with its final amendment in 1987 to incorporate the significant amount of earthquake engineering research performed in the preceding decade. This amendment replaced the entire earthquake resistant design section of the 1978 HBDB. In 1994 the HBDB was replaced after the abolishment of the MWD when Transit New Zealand published the first edition of the new Bridge Manual [29].

Both the 1987 HBDB and the 1994 Bridge Manual defined performance criteria for various levels of loading and represented the culmination of research projects in the areas of seismology and reinforced concrete behaviour. A new set of seismic zones and inelastic design spectra were incorporated along with in-depth recommendations for detailing plastic hinge zones and linkage systems.

Era 5 Seismic Loading

The 1987 HBDB introduced specific performance criteria for three different levels of loading. For the design earthquake, bridges were required to service emergency traffic and although some temporary repairs may have been required, reinstating the bridge to design level traffic and seismic capacities should have been feasible. For an earthquake with intensity significantly below the design earthquake, bridges were required to sustain only minor damage and remain serviceable with no disruption to traffic. During earthquakes with intensities significantly above the design level earthquake, collapse of the bridge was to be avoided even if damage was extensive. After temporary repairs the bridge was to be able to service emergency traffic and normal vehicle traffic was to be reinstated at a lower level of loading.

In the 1987 HBDB loading was defined based upon the type of structural action that the bridge was expected to exhibit. Structural action was defined as the bridge displacement at the centre of mass of the superstructure when subjected to an applied horizontal load and was used to define the level of global ductility in the bridge. Structural actions were classified as ductile structures, partially ductile structures, structures with limited ductility, or special cases. Ductile structures were to develop a ductility factor of six with post-yield displacement accompanied by an almost constant total resisting force. The ductility factor was limited to six to restrict damage during more frequent but less severe shaking. Plastic mechanisms were allowed to occur in either structural members or energy dissipating devices. Partially ductile structures were assumed to have the ability to form plastic mechanisms in some locations, but due to bridge geometry either the secondary members remained elastic or plastic hinging was incomplete at design level loading. Partially ductile structures were required to sustain a minimum ductility level of three. Structures with limited ductility described bridges which could not maintain a ductility factor of six under a design level earthquake either from lack of ductility demand or lack of ductility capacity. Bridges which were designed to remain elastic during seismic loading defined the upper limit of bridges that lacked ductility capacity.

Additionally, three special cases of structural action were described in the 1987 HBDB: structures on lead-rubber bearings, structures on rocking foundations, and structures “locked in” to the ground. Structures on lead-rubber bearings isolated the superstructure from ground accelerations through an isolation plane at the top of the piers. Structures on rocking foundations were either those that had shallow foundations that were sized to allow for uplift or were founded on deep foundations with sleeved piles. “Locked in” structures were bridges in which the structure remained elastic and moved with the surrounding ground. The superstructure was to rest on flexible piers and abutments with no allowance for relative displacement, and inertial forces were designed to be transmitted directly to the ground at the abutments. The use of “locked in” bridges was limited to a maximum length of 80 m, a maximum of three spans, and where ground stability at the abutments could be ensured during a design earthquake to allow for force transfer without gapping.

The 1987 HBDB further explained the application of these structural actions by providing maximum allowable ductility factors that could be used for various pier configurations and plastic hinge locations. These criteria are shown in Figure 3 and Table 4.

The base shear equation in the 1987 HBDB was updated from previous versions to reflect much of the work that the NRB Road Research Unit (RRU) had completed in the area of seismicity [30, 31]. The seismic base shear (H) in any horizontal direction was given by:

$$H = C_{\mu} Z W \qquad (2)$$

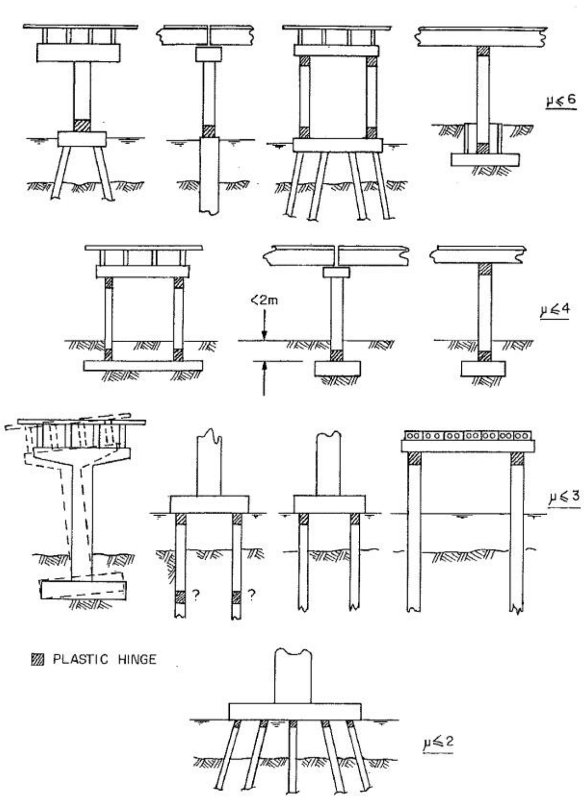

***Figure 3:** Examples of maximum values of μ allowed by Table 4 for various substructure configurations used from 1987 HBDB to 2013 Bridge Manual.*

Table 4. 1987 HBDB maximum allowable ductility factors.

Energy dissipation category	**μ**
Type D or Type P1 structures in which plastic hinges form above ground or water level	6
Type D or Type P1 structures in which plastic hinges form in reasonably accessible locations e.g. less than 2 m below ground but not below water level	4
Type D or Type P1 structures in which plastic hinges form in inaccessible locations or not at a level precisely predictable Type P2 structures Spread footings designed to rock	3
Hinging in raked piles in which earthquake load induces large axial forces	2
“Locked in” structures (T=0) Type L3 or elastic structures	1

Where:

C_μ = Basic horizontal force coefficient

Z = Return period coefficient

W = Weight of structure participating in response

The basic horizontal seismic coefficient (C_μ) was given by:

$$C_\mu = A_\mu \beta \geq 0.05 \tag{3}$$

Where:

A_μ = Lateral force coefficient from inelastic design spectra from Figure 4

β = Zone coefficient from Figure 5

A_μ defined the design spectra for structures with a ductility factor between one and six. The transition from a single inelastic spectrum for each zone with an assumed ductility of six in the early versions of the HBDB, to an inelastic design spectrum for each level of ductility, helped ensure that designers applied the appropriate base shear for each structural action type. Calculation of the fundamental period (T) of each principal axis of the bridge was based upon the combined stiffness of all supports in the direction being considered, using elastic material properties except in the case of reinforced concrete. For reinforced concrete members designed to yield, the stiffness of a cracked section at first yield over the full length of the member was used while the gross section stiffness was used for non-yielding members. When calculating fundamental period, bridge designers were to account for the effects of flexibility of foundations, bearings and variations in material properties. The mass considered to participate in the fundamental period was concentrated at the level of the superstructure centroid and equal to the mass of the superstructure, the pier caps (hammer heads) and half the mass of the piers.

The return period coefficient (Z) was based upon the importance level of the bridge and was not appreciably different from the importance factor (I) that was present in previous versions of the HBDB. Return periods ranged from 50-150 years with a maximum Z value corresponding to a 150 year return period (Table 5).

Clauses were provided in the 1987 HBDB to account for unintentional dynamic effects on the bridge during seismic loading. Seismic loading was to include additional forces and moments arising from accidental torsion caused by a 2.5% offset between the centre of mass and the centre of rigidity of the bridge in both horizontal directions. Designers were also instructed to account for additional moments caused by rotational inertia in single column hammer head piers.

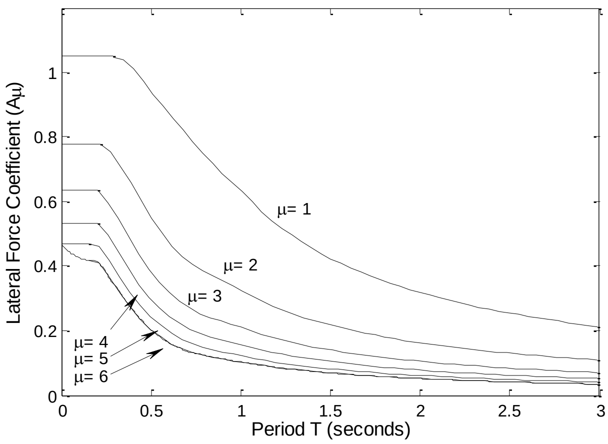

Figure 4: Basic seismic coefficient from 1987 MWD HBDB.

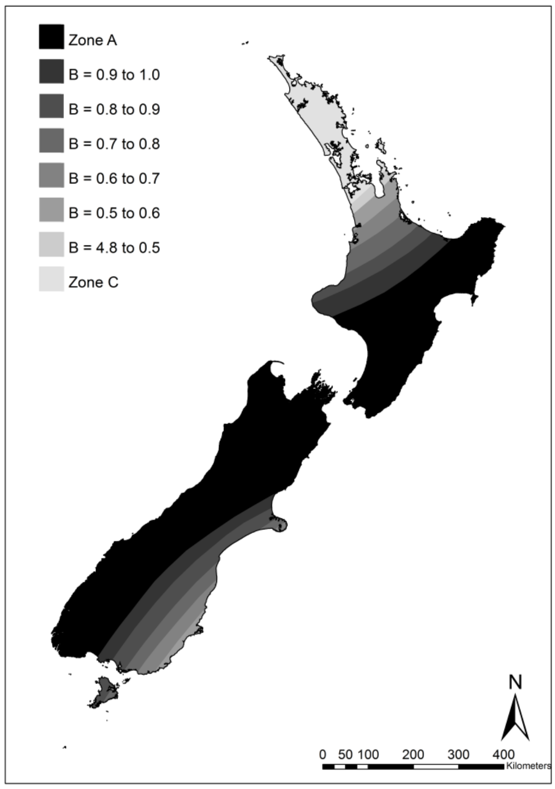

Figure 5: Seismic zones from 1987 HBDB.

Table 5. 1987 HBDB return period coefficient

Importance Category	Return Period (years)	Z
Bridges carrying more than 2,500 vehicles per day Bridges over or under motorway or railways. Bridge on national state highways	150	1
Bridges carrying between 250 and 2,500 vehicles per day Bridges on provincial state highways	100	0.88
Bridges carrying less than 250 vehicles per day and Non-permanent bridges	50	0.71

The 1994 Bridge Manual adopted the seismic provisions from the loadings code NZS 4203:1992. These provisions did not vary greatly from the 1987 HBDB other than a few minor updates to base shear calculation and some detailing requirements. However, because the MWD had at this time been privatized, there was a significant shift away from the prescriptive design procedures included in the HBDB's.

In the 1994 Bridge Manual the base shear calculation was updated to:

$$H = C_{\mu} ZRW \quad (4)$$

Where:

C_{μ} = basic horizontal force coefficient for normal soil sites (Figure 6) and for flexible soil sites (

Figure *7*)

Z = Zone Factor (Figure 8)

R = Risk factor

W = weight of structure participating in response

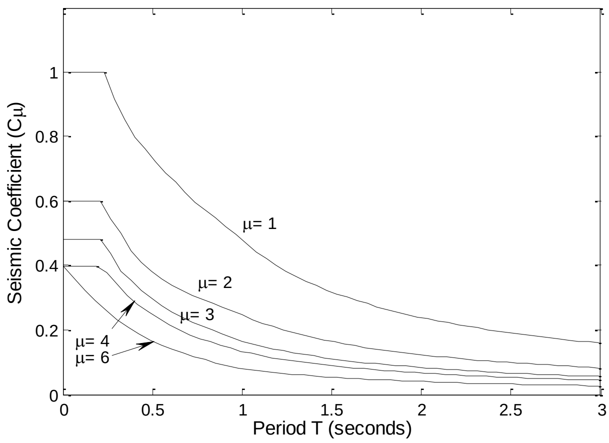

Figure 6: Basic seismic coefficient for normal soil sites from 1994 Bridge Manual.

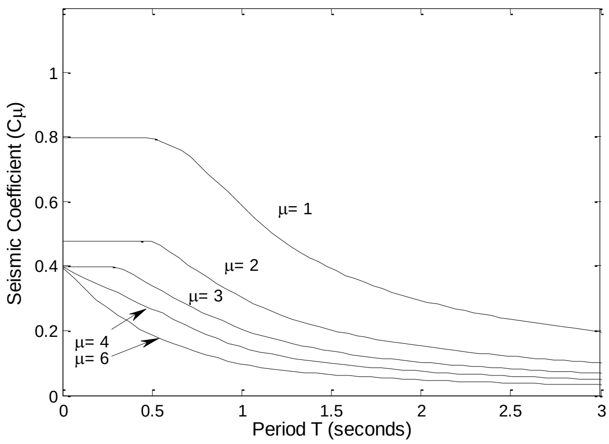

Figure 7: Basic seismic coefficient for flexible soil sites from 1994 Bridge Manual.

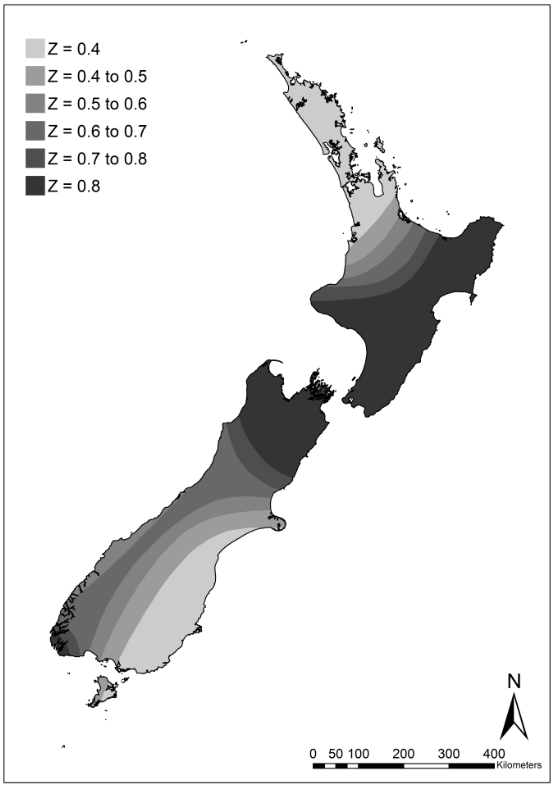

Figure 8: 1994 Transit Bridge Manual seismic zones.

This base shear calculation is essentially of the same form as the 1987 HBDB with the two equations from the HBDB combined into one equation and the return period and zone factor names changed. The return period doubled for high importance bridges to a 300 year return period and increased the risk factor values for bridges by 30% (Table 6). The basic seismic coefficient was updated to include spectra for bridges located on flexible soils. Limits for depths and strengths of cohesive and cohesionless soils were defined for flexible soil sites.

Table 6. Risk factor for 1994 Bridge Manual

Importance Category	Return Period (years)	R
Bridges carrying more than 2,500 vehicles per day Bridges over or under motorway or railways. Bridges on State Highway: 1, 2, 3, 3A, 4, 5, 6, 8, 8A	300	1.3
Bridges carrying between 250 and 2,500 vehicles per day Bridges on State Highways not listed above	200	1.1
Bridges carrying less than 250 vehicles per day and Non-permanent bridges	125	0.9

Only slight changes in structural action definitions were made to ductile structures and structures "locked in" to the ground while the definitions of other structural action types remained unchanged. The definition of ductile structures was clarified and it was required that a ductile structure be capable of sustaining a ductility factor of six through at least four cycles of loading with no more than 20% strength reduction. The prescriptive design methodology was removed but the capacity design principles in the seismic loading standard, NZS 4203:1992 [32] were cited. The definition of structures "locked in" to the ground was modified to remove the limits on what length and number of spans could be considered to behave "locked in." Structures were no longer assumed to be locked in for transverse earthquake loading unless a specific resisting system had been designed. The allowable ductility factors for various structural action types remained the same as those from the 1987 HBDB.

Era 5 Seismic Detailing

The 1987 HBDB provided member design processes for various lateral force resisting systems. Detailed methodologies were given to guide the design process for various methods used by bridges to resist loads. These methods included the incorporation of plastic hinges, having substructures that remain elastic, using Polytetrafluoroethylene (PTFE) coated steel sliding bearings, anchorage to a friction slab, behaving "locked in" or using mechanical energy dissipating devices. Bridges resisting earthquake loads through the development of plastic hinges were to be designed using capacity based design principles. The design of members resisting plastic hinge moments in these bridges was to be based on the overstrength flexural capacity of plastic members. Guidance on detailing reinforcement in plastic hinge zones was available in the concrete design standard of the time, NZS 3101:1982 [33]. The design of structures "locked in" to the ground required the ideal resistance of each abutment/soil system, in the respective direction, to be greater than the longitudinal design load from the whole bridge or the transverse load from half the bridge.

Notes on foundation design were also included in the 1987 HBDB amendment. When considering foundation stiffness for natural period calculation or to determine ductility demand, the designer was to account for the change in stiffness from liquefaction of the surface layers or the possibility of residual scour to a depth of two pile diameters below the pile cap. Pile foundations were to be designed in accordance with CDP 812: Pile Foundation Design Notes [34] to account for pile group actions and foundation strength based upon soil strength. Capacity design principles were used to determine the required strengths of piles, pile caps and the connections between these elements. The minimum required tensile strength between a pile and pile cap was 10% of the pile tensile strength, and a minimum region was to be reinforced for confinement for a plastic hinge, being the greater of one pile diameter or 450 mm below the pile cap. The contribution of the shell in steel shell piles was allowed to be included for shear and confinement but not for flexural strength unless adequate anchorage was provided. Raked piles were also required to be designed for both axial and flexural actions when subjected to seismic loading. This clause ended the practice of assuming that raked piles resisted only axial load when the line of action for a horizontal load coincided with the intersection of the axes of raked piles [22]. Requirements for friction slab design were also provided to ensure that a dependable horizontal resistance could be achieved from the approach fill.

Abutment requirements for the 1987 HBDB were similar to previous versions but the inertial forces from the self-weight of the abutment were included, the active earth pressure was replaced with the static earth pressure, and the greater of the passive or static earth pressure was used to design for backwall movement towards the approach fill. Design methods to ensure that abutments remained elastic under this loading were described in CDP 702/C: Retaining Wall Design Notes [24] and Matthewson *et al.* [35].

The requirements for horizontal linkages were expanded from previous versions of the HBDB. Horizontal linkages were defined as any positive linkage system such as shear keys, linkage bolts, or hinged deck slabs that minimized the possibility of span collapse during an earthquake. Linkage systems between the span and piers were mandated for simply supported spans. This requirement could be omitted at the abutments if the seat length was designed according to the provisions in this standard. This omission was allowed based upon the assumption that if greater seismic movement of the abutment occurred than was calculated, span unseating would not occur because abutment movement would be towards the bridge span due to the approach fill resisting movement away from the span [36]. Linkages were not required for continuous spans or for transverse restraint of simply supported spans if the requirements for minimum seat length were met. Elastomeric bearings with shear dowels were not allowed to be used as linkages.

The design of linkage systems was separated into two categories: tight linkages and loose linkages. Tight linkages described systems in which relative movement was not intended to occur during both service and design level seismic loading. Tight linkages were designed similarly to other seismic connections that transferred forces between spans. Where practical, rubber pads were to be added to tight linkage systems to allow for relative rotation of spans. Loose linkages allowed relative movement between elements and were used as a secondary precaution to avoid span collapse during an earthquake larger than the design level or if a pier top rotated due to uneven settlement. Rubber ring buffers were required in these systems, and the linkage system was considered not to be engaged until the ring buffers had compressed to half their original thickness. Both linkage systems were required to resist 20% of the dead load of the heavier adjacent spans as a minimum strength requirement.

Table 7. 1987 HBDB minimum seat length

Linkage System Type	Span/Support Overlap	Bearing Overlap
None	2.0E +100 mm (400 mm minimum)	1.25E
Loose Linkage	1.5E' +100 mm (200 mm minimum)	1.0E
Tight Linkage	200 mm minimum	N/A

Note: E = relative movement between span and support under combined earthquake and 1/3 of temperature movement

E' = relative movement at which the linkage system operates E'>E

Seat length requirements were also established corresponding to the type of linkage system used. The requirements for seat length (i.e. support overlap) and bearing overlap (i.e. distance from the centreline of the bearing to the edge of the support) are summarized in Table 7. For short skewed bridges these values were increased by 25% to account for the additional displacement caused by torsional behaviour of the bridge. Support types for standard bridge pier arrangements described in the MWD standard drawings [28] were also discussed and an interpretation of the codes used in the drawings to identify the seismic performance of various support types was provided. A preference was given to only using elastomeric bearings with the prescribed seat lengths for resistance in the transverse direction.

Few changes in seismic detailing from the 1987 HBDB were required with the publishing of the 1994 Bridge Manual. Foundation design requirements changed slightly with the requirement that designers consider pile flexure due to ground distortions and that the pile cap be able to resist vertical shear resulting from plastic hinging at pile tops. The ideal strength of the piles was to exceed the shear developed by the possible plastic mechanism at overstrength, but this clause could be neglected for partially ductile structures if this detail was judged to be too expensive by the designer. Abutments were designed using the recommendations provided by Wood and Elms [37].

Clauses were introduced that pertained to liquefaction and liquefaction-induced lateral spreading. For bridges located on flexible soil sites, an investigation of liquefaction potential of the site was required. Forces on the foundation due to soil deformation were determined, including the effects of soil stiffness. The designer was also to ensure that piles could sustain limited rotations from lower plastic hinges that could arise from liquefaction-induced lateral spreading.

Apart from the minimum seat length for loose linkage systems increasing from 200 mm to 300 mm, the linkage and seat length requirements from the 1987 HBDB were retained.

Era 6 (early-2000s to Present): Current Standards

In 2003 Transit New Zealand published the second edition of the Bridge Manual, which was amended in 2004 to include the seismic loading provisions from NZS 1170.5:2004. In 2013 the third edition of the Bridge Manual was published and is currently being used for bridge design.

Era 6 Seismic Loading

With the introduction of the 2003 Bridge Manual, most of the seismic loading provisions from the 1994 version were retained except for changes to horizontal loading and requirements for foundation design to resist the effects of soil liquefaction. Both the calculation of base shear and the seismic zones used to define seismic hazard throughout the country were updated. Base shear was calculated using the following equation:

$$V = C_{\mu} Z R S_p W_d \geq 0.05 W_d \tag{5}$$

Where:

C_{μ} = basic horizontal force coefficient

Z = Zone Factor

S_p = Structural performance factor

R = Risk factor

W_d = weight of structure participating in response

The basic horizontal force coefficient (C_{μ}) was defined for three separate subsoil condition categories: Category A (Figure 9), Category B (Figure 10), and Category C (Figure 11). Category A sites were either rock outcrops or stiff soil sites while Category C sites were constituted of flexible soils. Constraints for representative strengths and depths of cohesive and cohesionless soils defining either a Category A or Category C site were presented in the 2003 Bridge Manual (Table 8). Category B sites had intermediate soil stiffness and did not meet the requirements defining either Category A or C. The zone factor (Z) was updated with a new set of hazard contours (Figure 12). The return period used to calculate the maximum risk factor (R) was increased from 300 years to 1,000 years, but the value of the factor remained constant at 1.3 for bridges carrying more than 2,500 vehicles per day and only increased slightly for bridges of lower importance (Table 9).

Table 8. 2003 Bridge Manual Site Subsoil Category C criteria.

Soil Type and Description		Depth of soil (m)
Cohesive Soil	Representative undrained shear strengths (kPa)	
Soft	12.5 – 25	20
Firm	25 – 50	25
Stiff	50 – 100	40
Very Stiff	100 – 200	60
Cohesionless Soil	Representative SPT (N) values	
Loose	4 – 10	40
Medium Dense	10 – 30	45
Dense	30 – 50	55
Very Dense	>50	60
Gravels	>30	100

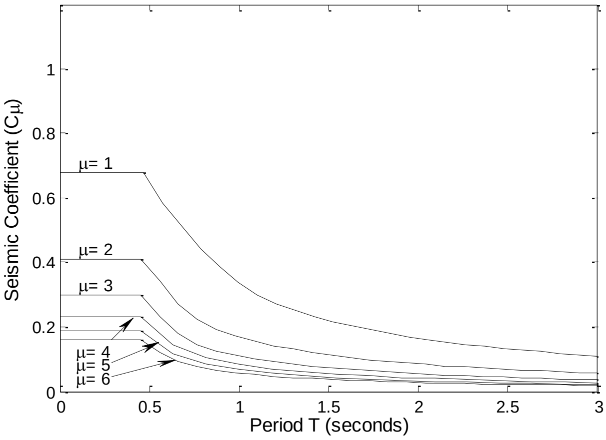

Figure 9: ***Basic seismic coefficient for very stiff soils or rock (Category A) from 2003 Bridge Manual.***

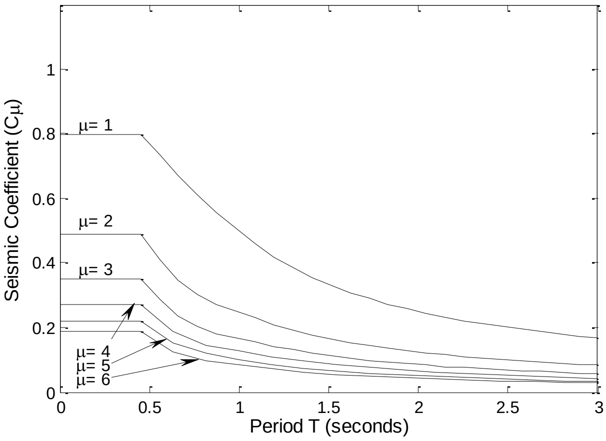

Figure 10: ***Basic seismic coefficient for intermediate soils (Category B) from 2003 Bridge Manual.***

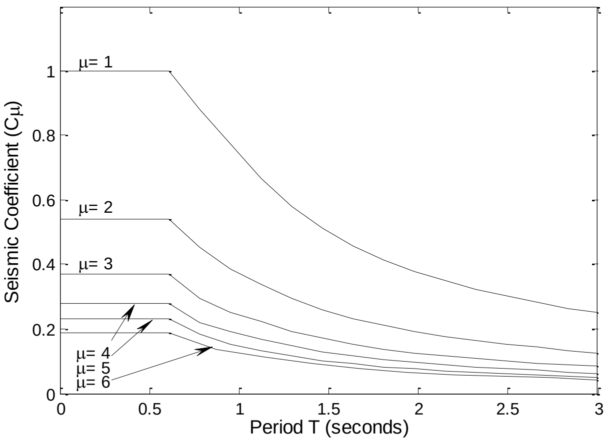

Figure 11: ***Basic seismic coefficient for flexible or deep soils (Category C) from 2003 Bridge Manual.***

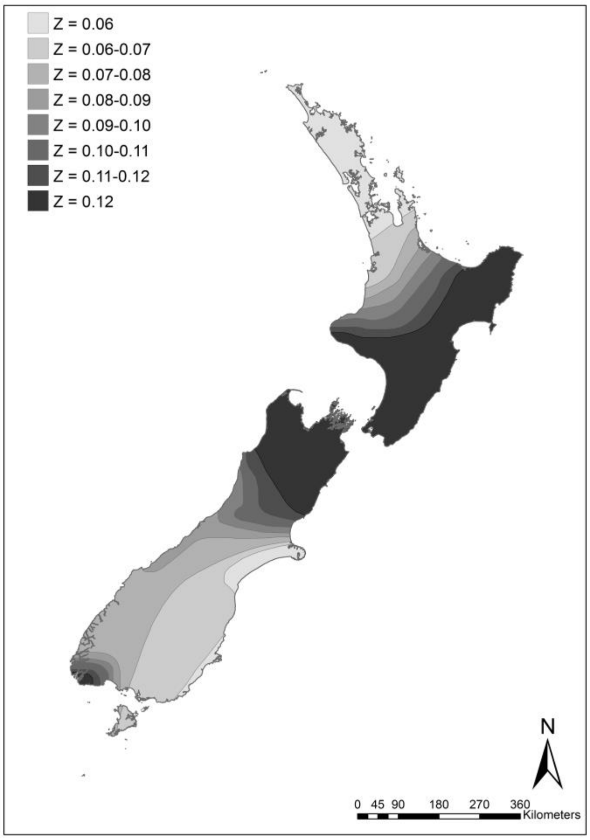

Figure 12: ***Seismic zones from 2003 Bridge Manual.***

Table 9. Risk factor for 2003 Bridge Manual

Importance Category	Return Period (years)	R
Bridges carrying more than 2,500 vpd Bridges over or under motorway or railways. Bridges on State Highway: 1, 2, 3, 3A, 4, 5, 6, 8, 8A	1,000	1.3
Bridges carrying between 250 and 2,500 vpd Bridges on State Highways not listed above	450	1.15
Bridges carrying less than 250 vpd Non-permanent bridges	350	1.0

Two factors in the 2003 Bridge Manual were used to account for behaviour not addressed in previous bridge standards. The structural performance factor (S_p), originally introduced in Amendment 1 of the June 1995 Bridge Manual, was used to reduce base shear to account for the effects of additional damping provided at foundations and abutments. An increase in design earthquake loading was also suggested for sites in near fault regions if the cost of bridge replacement in the event of significant damage was expected to be high. Additionally, orthogonal earthquake effects were accounted for by combining the absolute values of forces and moments resulting from loading from 100% of the direction being considered (e.g. transverse) and 30% of the loading from the orthogonal direction (e.g. longitudinal).

In 2004 the 2nd edition of the Bridge Manual was amended to include the updated base shear calculations and seismic zones introduced in NZS 1170.5:2004 [38]. These updates were also included in the 2013 edition of the Bridge Manual. For the structural performance criteria, limits were provided to define what constituted earthquakes above and below the design level. Earthquakes below design level were considered to be those resulting from a return period one fourth that of the design level while earthquakes above design level were to result from a return period one and a half times larger than the design level. The return period was increased to 2,500 years for all bridges on primary transportation routes (i.e. importance level 3 and 4) and to 1,000 years for ordinary bridges (i.e. importance level 2).

Base shear was calculated using:

$$V = C_d(T_1)W_t \tag{6}$$

Where:

$C_d(T_1)$ = Horizontal design action coefficient

W_t = Weight of structure participating in the response in the direction being considered

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu} \geq \left(\frac{Z}{20} + 0.02\right)R_\mu \tag{7}$$

but not less than 0.03 R_u

Where:

$C_d(T_1)$ = From elastic site hazard spectrum, determined for the natural period of vibration

T_1 = Fundamental period of vibration

S_p = Structural performance factor

Z = Hazard factor

R_u = Return period factor

k_μ = Modification factor for ductility

For soil classes A-D:

$$k_\mu = \mu \quad \text{for } T_1 \geq 0.7 \text{ seconds} \tag{8a}$$

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1 \quad \text{for } T_1 < 0.7 \text{ seconds} \tag{8b}$$

For soil classes E:

$$k_\mu = \mu \quad \text{for } T_1 \geq 1.0 \text{ seconds} \tag{9a}$$

$$k_\mu = (\mu - 1.5)T_1 + 1.5$$
$$\text{for } T_1 < 1.0 \text{ seconds and } \mu \geq 1.5 \tag{9b}$$

$$C(T_1) = C_h(T_1)ZRN(T,D) \tag{10}$$

Where:

$C_h(T_1)$ = Spectral shape factor from Figure 13,

T_1 = Fundamental period of vibration,

Z = Hazard factor,

R = Return period factor R_s or R_u for the appropriate limit state of serviceability or ultimate respectively. Limited such that $0.13 \leq ZR_u \leq 0.7$,

$N(T,D)$ = Near fault factor.

Curves for the basic horizontal force coefficient (C_h) are defined for five separate subsoil condition categories, Classes A-E. Representative strengths and depths of cohesive and cohesionless soils, site natural periods, and average shear wave velocities over the top 30 m are defined for all categories in NZS 1170.5:2004. The structural performance factor (S_p) is equal to 0.9 for Class A and B, 0.8 for Class C, and 0.7 Class D or E. The zone factor (Z) is determined from NZS 1170.5:2004 (Figure 14), except in the Northland and Canterbury regions. Bridges located in Northland are to use the contours provided in the 2013 Bridge Manual (Figure 15), while Canterbury bridges are to use the NZ Building Code verification method B1/VM1.

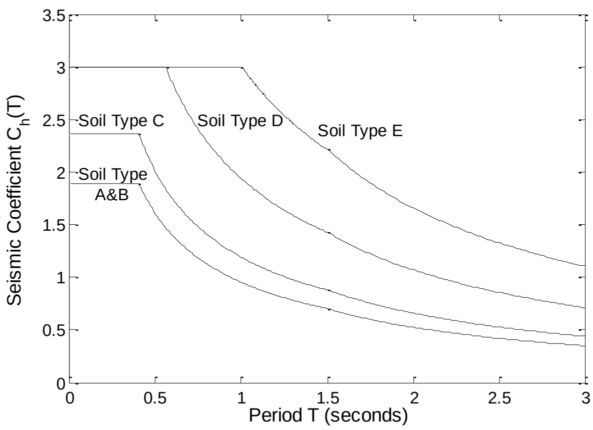

Figure 13: Seismic coefficient for various site classes from NZS 1170.5.

Era 6 Seismic Detailing

The only update to the seismic detailing requirements in the 2003 Bridge Manual was to foundation design, primarily with regards to the effects of liquefaction. The requirements for the mitigation of liquefaction effects were expanded from the 1994 Bridge Manual, with liquefaction potential assessments performed using the methods presented in the NCEER Workshop on Evaluating Liquefaction Resistance of Soils [39]. In the event of liquefaction occurring at the site, designers were to account for the possibility of foundation failure, loss of pile vertical or lateral load capacities, subsidence, down-drag on piles and lateral spreading of ground. An emphasis was also placed on considering how the effects of liquefaction at the bridge site might impact the surrounding area and the transit route. These criteria were substantially expanded in the 2013 Bridge Manual to include hazard maps to determine peak ground acceleration for a given magnitude earthquake, providing an explanation of what types of soils are susceptible to liquefaction, guidance on site investigations, guidance on what methods to use for liquefaction assessment and what methods to use to determine the effects of liquefaction on the bridge. Sections were also provided for guidance on how to mitigate the effects of liquefaction and how to optimise the ground improvement programme.

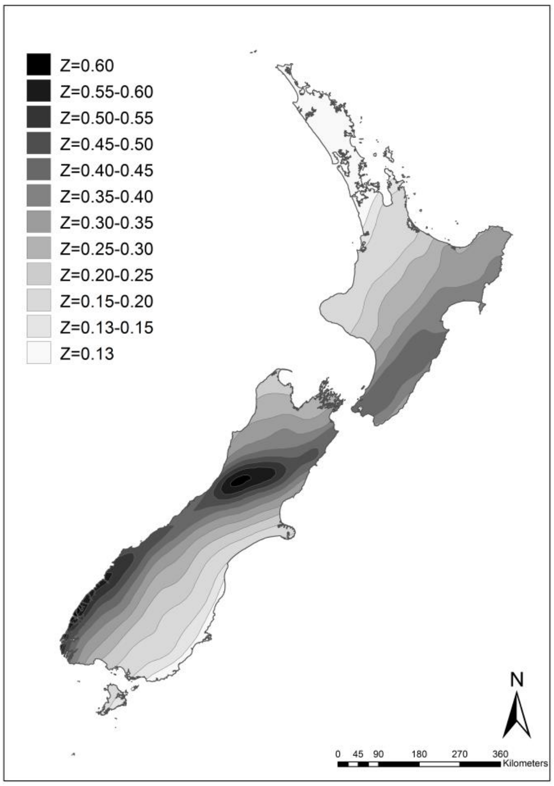

Figure 14: Seismic zones from NZS 1170.5: 2004.

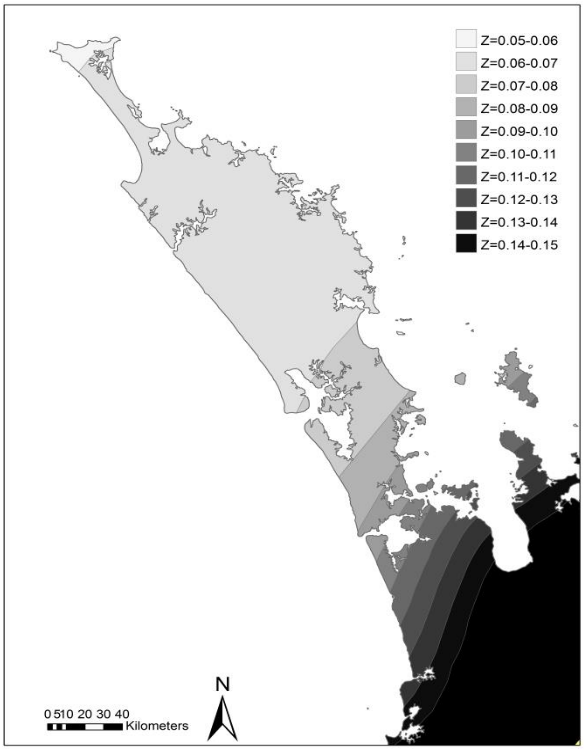

Figure 15: Seismic zones for Northland from 2013 Bridge Manual.

The 2013 Bridge Manual also expanded the provisions for linkage systems by providing requirements for linkage bars in Appendix C. Requirements for bar and rubber pad material, geometry, and corrosion resistance were provided along with loading requirements for both serviceability and ultimate limit states.

SUMMARY AND COMPARISON OF STANDARDS

In order to equate the seismic performance of bridges constructed during different eras, design requirements for each era are summarised and compared. Comparisons are made for seismic loading of different eras while major changes are summarised for detailing for ductility, foundation design, and linkage systems.

Comparison of Loading

The base shear coefficients of different eras were compared to the current level of loading derived from the 2013 Bridge Manual. The ratios of loadings at each era to the 2013 Bridge Manual were computed for two different periods, at four different ductility levels, and for Soil Type C at six different geographic locations (Table 10). The period of T = 0.5 s was used as most bridges in New Zealand have short periods and design spectra used from Era 2 to Era 6 have constant spectral accelerations up to approximately 0.5 s. Therefore, base shear coefficients for 0.5 s are indicative of most bridges in New Zealand even if those bridges have fundamental periods less than 0.5 s. The period of T = 1.0 s was used to show the change in base shear coefficients for longer period bridges. Base shear coefficient ratios were only computed using return periods for bridges which were defined by the high importance category in each design era. Base shear coefficient ratios for other return periods can be obtained by scaling the results in Table 10 by the appropriate importance/risk factor defined in the previous sections. As Table 10 only provides the change in base shear coefficients for six geographic locations, a generalized depiction of base shear coefficient changes with respect to design era for all of New Zealand is shown in

Figure ***16*** and

Figure ***17***.

As Table 10 only provides base shear coefficient ratios between different design eras, only a comparison of seismic demand could be made between each design era. No comparison was made between bridge capacities of a given design era because seismic capacity is a bridge specific value and therefore an inappropriate measure to use for comparing all bridges within one design era to another. It should be noted that when comparing Era 2 and Era 3 bridges to current practice, the flexural capacities determined using working stress design will cause the section to yield at between 1.3 to 1.5 times higher loading than the design base shear. Further information with regards to comparing section capacities designed using working stress to current practice can be found in Fenwick and MacRae [9].

When comparing base shear coefficients from different eras several factors need to be considered. The primary factor affecting base shear calculations is the variation of how stiffness of concrete sections was computed when determining fundamental period. Early standards used the gross section, while subsequent standards used a percentage of the moment of inertia of the concrete section to account for cracked section behaviour. Fenwick and MacRae [9] computed shifts in fundamental period for historical concrete standards used during the design eras defined here. After adjusting these shifted periods to only account for column cracking, the

Table 10. Base Shear Coefficients from Different Design Eras as Ratio of Current Loading Criteria.

City	Ductility	Period (s)	Era 2* 1933-1964	Era 3* 1965-1975	Era 4 1976-1986	Era 5 1987-1993	Era 5 1994-2002	Era 6 2003	Era 6 2004-present
Auckland	μ = 1	T = 0.5	0.35	0.28	0.28	1.71	1.35	1.67	1.0
		T = 1.0	0.58	0.41	0.37	1.88	1.46	1.82	1.0
	μ = 2	T = 0.5		0.48	0.48	1.82	1.21	1.67	1.0
		T = 1.0		0.82	0.75	1.96	1.52	1.82	1.0
	μ = 4	T = 0.5		0.87	0.87	1.93	1.42	1.70	1.0
		T = 1.0		1.56	1.42	1.81	1.62	1.80	1.0
	μ = 6	T = 0.5			1.27	1.98	1.32	1.68	1.0
		T = 1.0			1.42	1.17	0.92	1.18	1.0
New Plymouth	μ = 1	T = 0.5	0.19	0.15	0.15	1.66	1.32	1.54	1.0
		T = 1.0	0.32	0.21	0.20	1.82	1.42	1.69	1.0
	μ = 2	T = 0.5		0.26	0.26	1.77	1.17	1.55	1.0
		T = 1.0		0.43	0.41	1.91	1.48	1.69	1.0
	μ = 4	T = 0.5		0.49	0.49	1.88	1.38	1.58	1.0
		T = 1.0		0.86	0.82	1.85	1.65	1.75	1.0
	μ = 6	T = 0.5			0.71	1.93	1.28	1.56	1.0
		T = 1.0			1.21	1.77	1.39	1.69	1.0
Napier	μ = 1	T = 0.5	0.09	0.15	0.15	0.94	0.71	0.88	1.0
		T = 1.0	0.15	0.20	0.20	1.03	0.77	0.96	1.0
	μ = 2	T = 0.5		0.25	0.25	1.0	0.64	0.88	1.0
		T = 1.0		0.40	0.39	1.08	0.80	0.96	1.0
	μ = 4	T = 0.5		0.46	0.46	1.06	0.75	0.90	1.0
		T = 1.0		0.80	0.78	1.04	0.89	1.0	1.0
	μ = 6	T = 0.5			0.67	1.09	0.70	0.89	1.0
		T = 1.0			1.17	1.01	0.77	0.98	1.0

***** It should be noted that due to working stress design methods the flexural capacities for Era 2 and 3 bridges will be 1.3 to 1.5 times higher than the equivalent ultimate strength flexural capacities of bridges designed in other eras. This increase in flexural strength should be accounted for during assessment of the seismic capacity of bridges designed during this era.

adjusted periods were used to adjust the calculated base shear ratios calculated in Table 10. Another significant factor was that often foundation flexibility was not included in the calculation of structural periods in Eras 2 and 3.

The level of loading for different global ductility factors was also considered for Eras 2, 3, and 4. Base shear coefficient ratios were computed for global ductility factors of 1, 2, 4 and 6 for these eras but it is doubtful that this range of ductility was achieved in each of these eras. Era 2 bridges were designed to remain elastic under 0.1g and it is doubtful that significant post-yield displacement could be achieved and therefore base shear coefficients for Era 2 bridges with ductility levels above one have been omitted from Table 10. However, it should be noted that many bridges constructed during this era, especially short bridges with wall type piers, are likely to have seismic capacity that exceeds the 0.1g design load due to the additional capacity from working stress design methods and damping that exceeds the nominal 5% value for which they were designed. Design base shear demand for bridges constructed in Era 3 and Era 4 was based upon the bridges reaching a target ductility. While there were guidelines on what level of ductility was acceptable if the target ductility was not met, there was no explicit guidance on increasing the loading for lower levels of ductility. While some designers did increase the base shear to the appropriate level of ductility (e.g. for an elastic structure the base shear provided for a target ductility of six would be multiplied by six), it is not clear that this practice was widespread. Therefore, the base shear ratios for Era 3 and Era 4 in Table 10 are computed for each ductility level based upon the load at the target ductility as a lower bound for comparisons. Additionally, base shear coefficient ratios for Era 3 bridges with a global ductility of six are omitted from Table 10 due to the lack of guidance on detailing of reinforcement to achieve this level of ductility.

The base shear coefficient comparisons do not include any adjustment for the age of the bridge or different site classes. The increase in concrete strength or the deterioration of

Table 10 Continued:

City	Ductility	Period (s)	Era 2* 1933-1964	Era 3* 1965-1975	Era 4 1976-1986	Era 5 1987-1993	Era 5 1994-2002	Era 6 2003	Era 6 2004-present
Wellington	μ = 1	T = 0.5	0.09	0.14	0.14	0.92	0.70	0.86	1.0
		T = 1.0	0.15	0.20	0.19	1.0	0.75	0.94	1.0
	μ = 2	T = 0.5		0.25	0.25	0.98	0.62	0.86	1.0
		T = 1.0		0.39	0.38	1.05	0.78	0.94	1.0
	μ = 4	T = 0.5		0.45	0.45	1.04	0.73	0.88	1.0
		T = 1.0		0.78	0.76	1.02	0.87	0.97	1.0
	μ = 6	T = 0.5			0.65	1.06	0.68	0.87	1.0
		T = 1.0			1.15	0.99	0.75	0.96	1.0
Christchurch	μ = 1	T = 0.5	0.12	0.14	0.14	1.00	0.56	0.74	1.00
		T = 1.0	0.19	0.20	0.19	1.09	0.61	0.81	1.00
	μ = 2	T = 0.5		0.24	0.24	1.06	0.50	0.74	1.00
		T = 1.0		0.39	0.37	1.14	0.63	0.81	1.00
	μ = 4	T = 0.5		0.44	0.44	1.13	0.59	0.76	1.00
		T = 1.0		0.78	0.75	1.11	0.71	0.84	1.00
	μ = 6	T = 0.5			0.63	1.16	0.55	0.75	1.00
		T = 1.0			1.12	1.08	0.61	0.83	1.00
Greymouth	μ = 1	T = 0.5	0.09	0.15	0.15	0.96	0.55	0.68	1.0
		T = 1.0	0.16	0.21	0.20	1.06	0.59	0.74	1.0
	μ = 2	T = 0.5		0.26	0.26	1.03	0.49	0.68	1.0
		T = 1.0		0.41	0.40	1.10	0.62	0.74	1.0
	μ = 4	T = 0.5		0.47	0.47	1.09	0.58	0.69	1.0
		T = 1.0		0.82	0.80	1.07	0.69	0.77	1.0
	μ = 6	T = 0.5			0.69	1.12	0.54	0.68	1.0
		T = 1.0			1.20	1.04	0.59	0.75	1.0

***** It should be noted that due to working stress design methods the flexural capacities for Era 2 and 3 bridges will be 1.3 to 1.5 times higher than the equivalent ultimate strength flexural capacities of bridges designed in other eras. This increase in flexural strength should be accounted for during assessment of the seismic capacity of bridges designed during this era.

section capacity due to corrosion as a given bridge ages were not included in the base shear comparisons because these factors are site specific and are inappropriate for this generalized comparison [40]. Base shear coefficients were only computed for bridges located on soils equivalent to Site Class C as defined in NZS 1170.5:2004. This limitation to one site class was to allow for consistent comparison for all eras as prior to the 1987 HBDB no separate design spectra were provided for flexible soil sites. If compared to flexible soil sites, base shear coefficient ratios would be expected to be 25-30% lower due to the increase in spectral acceleration for soft soil sites.

Base shear coefficients in Table 10 are presented in Figure 18 for areas of similar seismicity. Low seismicity areas were represented by Auckland, medium seismicity areas by New Plymouth and Christchurch, and areas of high seismicity were represented by Napier, Wellington, and Greymouth. Base shear ratios presented in Figure 18 for a given level of seismicity are an average of the ratios presented in Table 10 for a given era, ductility and period for the locations used to represent that seismicity level. Figure 18 shows that the base shear ratios of previous standards are not all lower than those used in the 2013 Bridge Manual. In all areas of seismicity base shear coefficients increased until the 1987 HBDB, then decreased with the publication of the 1994 Bridge Manual, and finally increased again with the publication of both the 2003 and 2013 Bridge Manuals. Because the base shear coefficients used since the 1987 HBDB for areas of low to medium seismicity are between 100 - 200% of current design levels for both 0.5 s and 1.0 s periods and considering that detailing was similar to current practice, bridges subjected to ground shaking in these areas and designed since 1987 are expected to behave similarly to those bridges designed to the 2013 Bridge Manual. This behaviour is expected for bridges designed using either the 1987 HBDB or the 2002 Bridge Manual in areas of high seismicity, such as Napier or Wellington, but not for bridges designed using the 1994 Bridge Manual as those base shear coefficients are as low as 60% of current levels as a result of the use of a 300 year return period used during that time instead of the 2,500 year return period used currently. This lower loading for bridges designed using the 1994 Bridge Manual may result in bridges that lack shear and displacement capacity.

During Era 3 and 4, base shear coefficients for the minimum target ductility of four are between 45 - 85% of current levels for 0.5 s period bridges and between 80 - 150% for 1.0 s period bridges depending upon seismic zone. But if the bridge was not detailed adequately to achieve a ductility of four, as is likely for Era 3 bridges due to the lack of plastic hinge detailing guidelines, base shear coefficient ratios reduce approximately linearly with reduced ductility. For example, if a bridge with a fundamental period of 1.0 s was constructed in an area of high seismicity during Era 3, it would be designed for a base shear assuming a ductility of four. But if the detailing was such that this bridge could only achieve a global ductility of two, the base shear coefficient ratio would reduce from 80% of current code to 40%. This reduced base shear coefficient ratio can be as low as 25% for short period bridges in areas of high seismicity, highlighting the need for detailed assessment.

As Era 2 bridges were designed to respond elastically when subjected to seismic loading, comparisons were made to the elastic spectra, i.e. with a global ductility equal to one. Era 2 bridges with fundamental periods of 0.5 s and 1.0 s built in areas of low seismicity were 35% or 58% of the 2013 Bridge Manual base shear coefficients respectively. The percentage of base shear coefficients was reduced to 9% for T = 0.5 s and to 15% for T = 1.0 s if the bridges were built in areas of high seismicity. While these base shear coefficient ratios are significantly below current levels, it should be noted that many bridges built during Era 2 have been found to perform well in a design level earthquake [41, 42].

Comparison of Detailing for Ductility

Bridges in the New Zealand bridge stock were designed to respond elastically to seismic loads until the late 1960s. In the early 1970s seismic bridge design moved towards ductile design with the introduction of plastic hinge detailing requirements such as the use of likely material strengths instead of lower bound strengths, allowance for overstrength, and prohibiting the use of high strength brittle reinforcing steels in plastic hinge zones.

In the mid-1970s the widespread use of capacity design and detailing for ductile response began. In-depth instruction on detailing to achieve a given ductility was provided with the publication of CDP 810/A: Ductility of Bridges with Reinforced Concrete Piers. By the 1980s capacity based design and detailing for ductility had become codified in both NZS 3101:1982 and the 1987 HBDB. The publication of the 1987 HBDB also introduced the use of different loadings for different levels of ductility as well as guidance on to what ductility various structural configurations should be designed. The codification of ductile detailing since the 1980s suggests that bridges built during this era would perform well if subjected to ground shaking.

Comparison of Foundation Detailing

Foundation seismic detailing requirements were introduced in Era 2 with the 1956 Bridge Manual. Pile foundations were typically not designed to resist lateral loads unless they were raked. This practice continued until the 1960s. Abutments were only designed to resist inertial forces from their own self-weight and the supported bridge spans, with seismic earth pressures not included in abutment design until Era 3. Abutments designed using Era 3 and Era 4 code provisions were designed to resist earthquake induced earth pressures, active and passive earth pressures, and superstructure inertial loads while ignoring the inertial load from the abutment self-weight.

Requirements that piles be capable of forming plastic hinges under the pile cap were introduced in the 1987 HBDB by setting specifications for confinement and minimum strengths between the pile and pile cap. Designers were also to account for foundation strength reduction from residual scour and group action. Abutment design requirements were updated to include inertial forces from the self-weight of the abutment and to replace the active earth pressure with the static (at-rest) earth pressure. Abutments were to remain elastic under this loading.

The 1994 Bridge Manual introduced requirements to design bridge foundations to resist induced forces from liquefaction and liquefaction-induced lateral spreading, and required a site investigation to determine liquefaction potential. These requirements were updated by the 2003 Bridge Manual, which provided guidance on the determination of liquefaction potential for a site and what failure mechanisms to consider if liquefaction occurred. These guidelines were substantially expanded in the 2013 Bridge Manual by providing more guidance on liquefaction potential assessment, induced loads, and mitigation options. Because such detailed design requirements for liquefaction were only recently published it is expected that if no ground improvement or other mitigation techniques were implemented even bridges constructed at the end of Era 5 are likely to sustain abutment and approach damage if constructed on liquefiable sites, as was observed during the Canterbury earthquake sequence [41, 42].

Comparison of Linkage Systems and Seat Lengths

Linkage systems were introduced in 1956 in response to the increased use of steel and precast concrete superstructures [15]. Loading for these linkage systems was updated in Era 3 but no requirements for seat length were introduced until the 1987 HBDB. The 1987 HBDB classified linkage systems as either tight or loose linkage types and set design requirements and seat lengths for each type. Seat lengths were to be increased for skewed bridges.

The 1994 Bridge Manual increased the required seat length for loose linkage systems, but design requirements remained otherwise unchanged until requirements for linkage bar material, geometry, corrosion resistance and loading for both serviceability and ultimate limit states were introduced in the 2013 Bridge Manual.

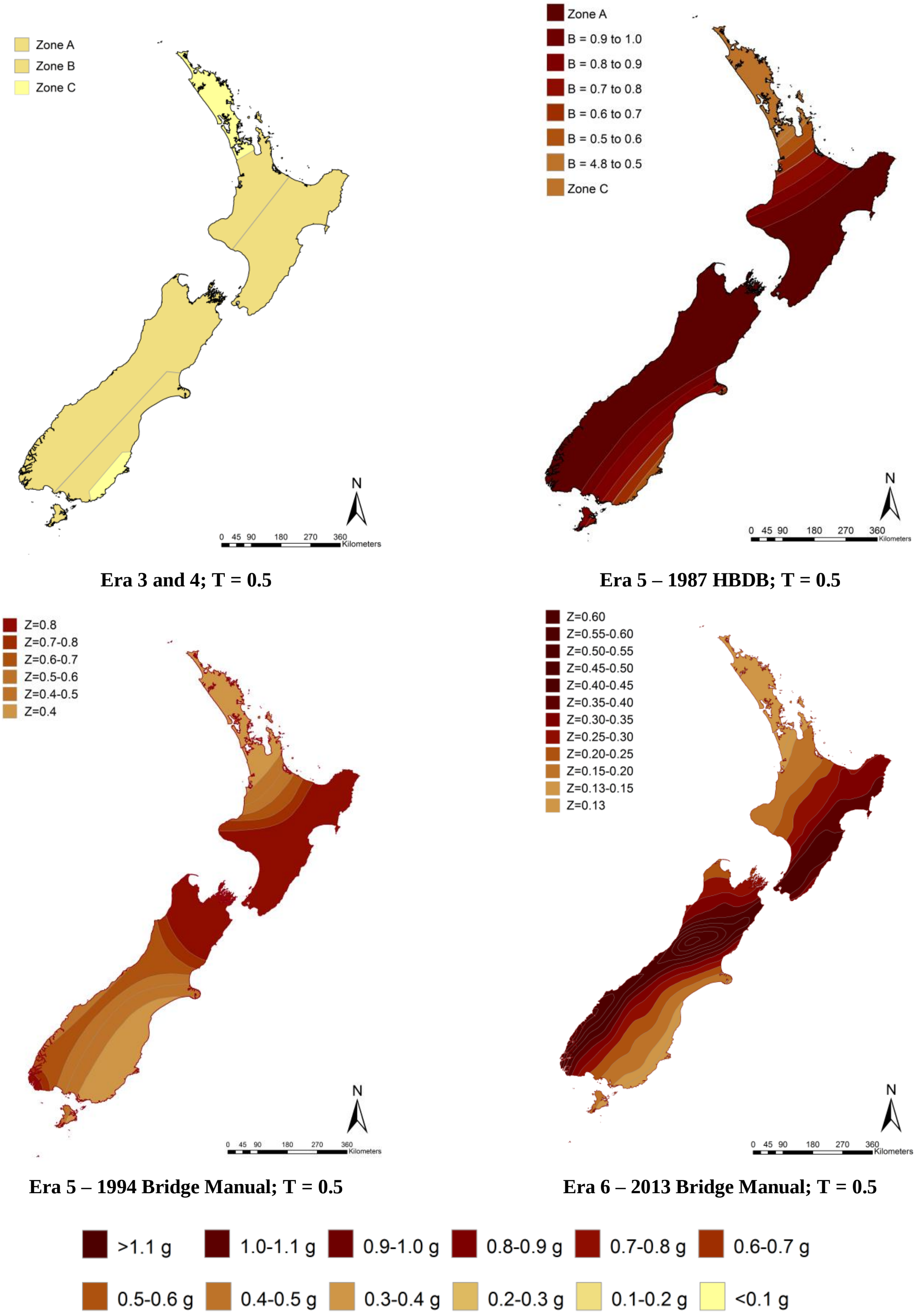

Figure 16: Base shear coefficient for bridges with T = 0.5 s period and μ =1 for Site Class C or equivalent and highest importance/risk factor. Era 2 maps are not shown as all coefficients are equal to 0.1g.

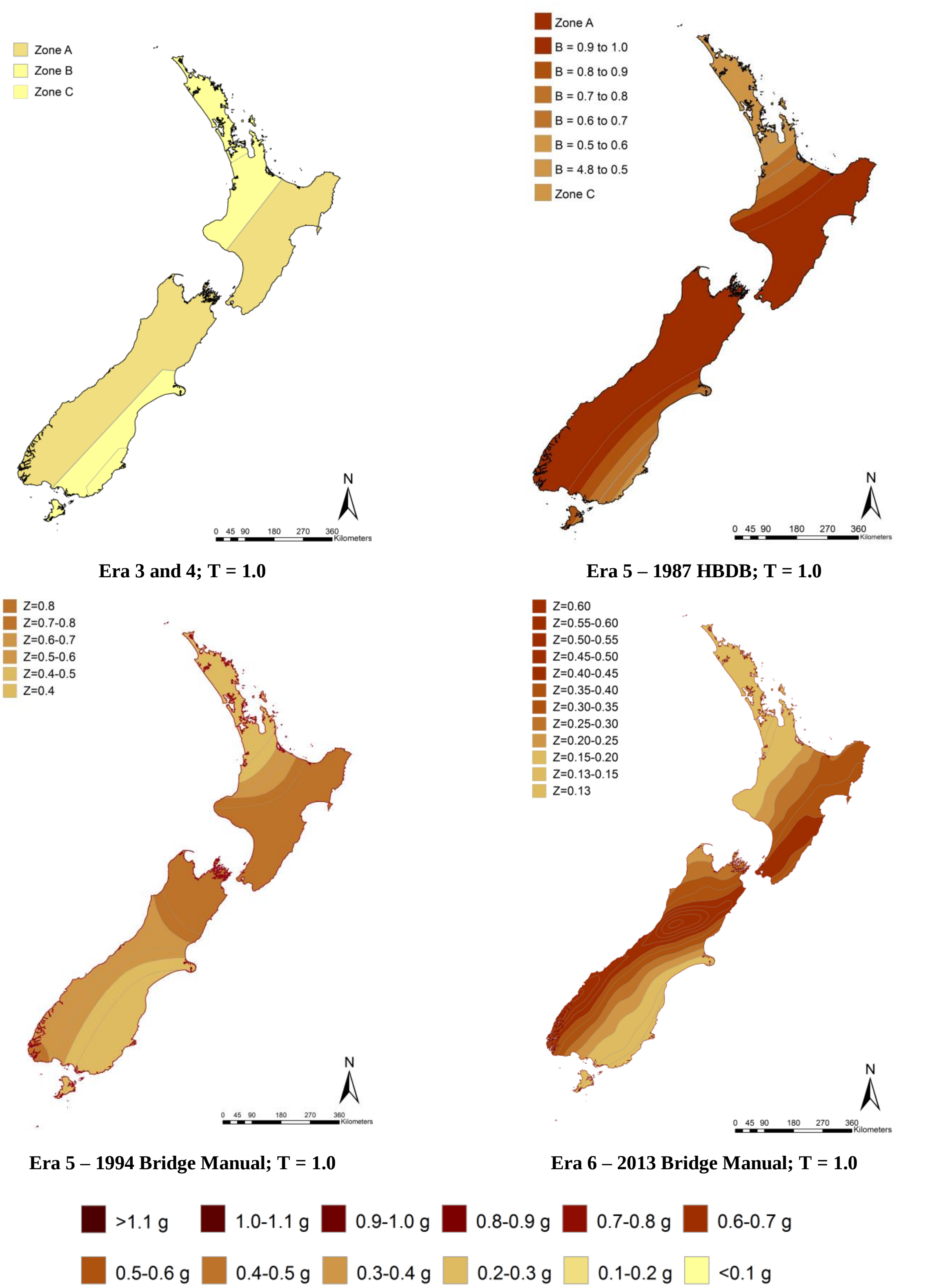

Figure 17: Base shear coefficient for bridges with T = 1.0 s period and μ =1 for Site Class C or equivalent and highest importance/risk factor. Era 2 maps are not shown as all coefficients are equal to 0.1g.

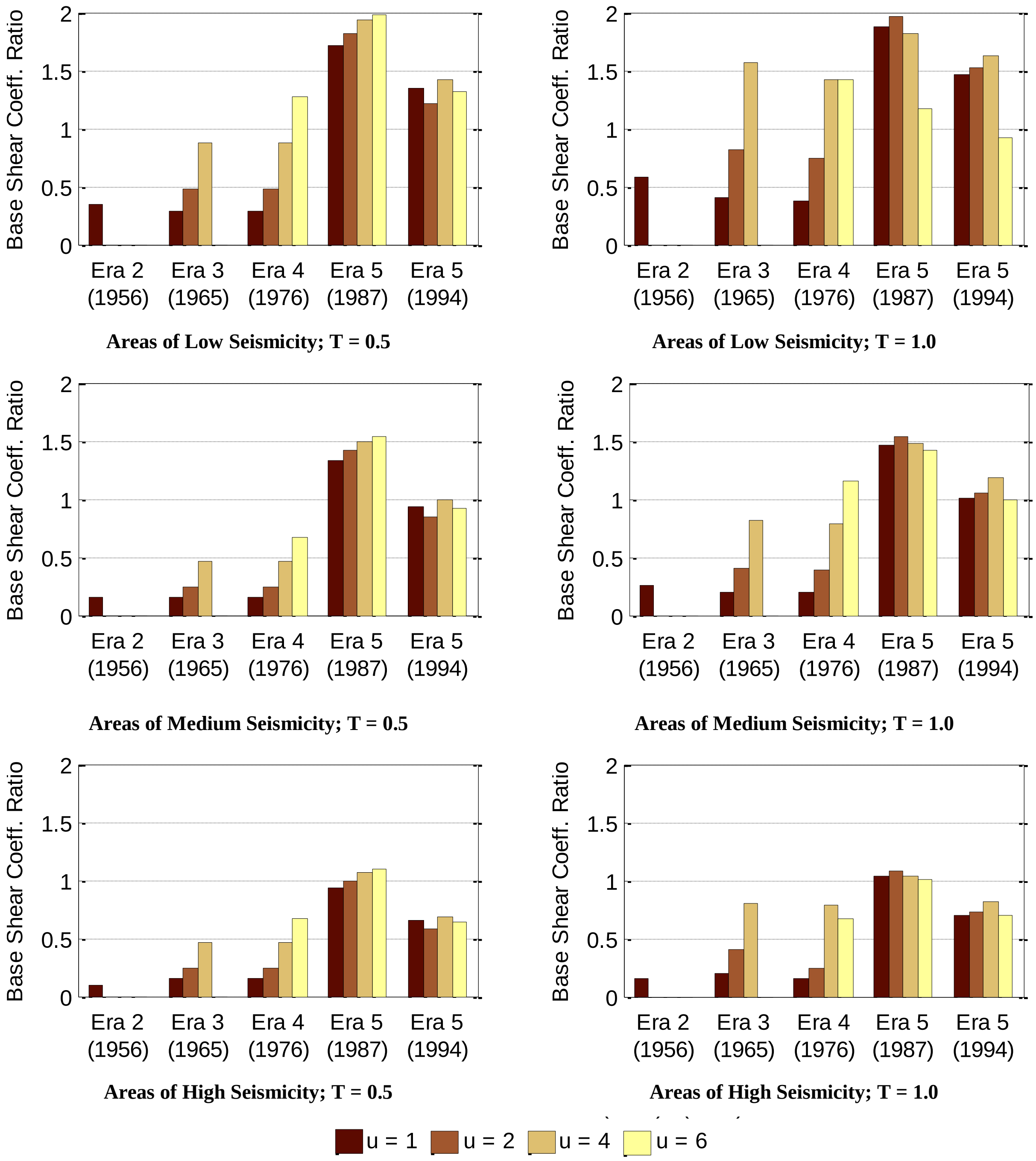

Figure 18: ***Base shear coefficient ratios as compared to the 2013 Bridge Manual for areas of high, medium and low seismicity for Site Class C or equivalent and highest importance/risk factor.***

CONCLUSIONS

For much of the twentieth century bridge design was controlled by a single organization operating under the various names of PWD, MoW, or MWD, until bridge design was privatised in 1988. Such a long oversight tenure by a single entity allowed for a high degree of control on bridge design through published design standards, and an efficient dissemination of best practice design procedures through supplementary design guides. A review of seismic bridge standards revealed the major changes in seismic loading and detailing. Seismic standards were introduced in 1933 and required all bridges to be designed to resist a lateral force of 0.1g until the introduction of seismic zones and design spectra in 1965. While Era 1 and 2 bridges are expected to have been poorly detailed for ductile behaviour and were designed to as low as 9% of current base shear levels, collapse of spans is unlikely in many cases due to a strong preference to design monolithic structures and the likelihood of seismic capacity above design levels due to structural configuration. However there were numerous structures built in this era, such as arch bridges with vulnerable arch and spandrel column members, and bridges with wall-type piers supported by lightly reinforced piles that extend well above ground level due to scour. The prevalence of these potentially vulnerable structures and the lower levels of base shear to which they were designed, accentuates the need to prioritise their seismic capacity assessment for a given road network.

Capacity based design requirements were introduced in 1971 and were refined through a series of design guidelines through the 1970s and 1980s. Bridges constructed during the first half of this era should be evaluated to ensure that detailing was adequate to achieve the design ductility of four, as ductile detailing requirements were not well established at that time.

The basis of the current bridge design standard was established with the 1987 HBDB, which provided updated seismic hazard zones, design spectra, considerations for flexible soils, established ductility levels for various structural action types and introduced new requirements for foundation and linkage system detailing. Subsequent bridge standards further refined the calculation of base shear and placed a higher emphasis on accounting for the effects of liquefaction on a bridge. Bridges designed from 1987 onwards are expected to perform well when subjected to ground shaking as ductile design had matured, but bridges constructed from the mid to late 1990s in areas of high seismicity should be checked to ensure that they possess adequate capacity due to a 30 to 40% reduction in base shear compared to current levels. Unless bridges were constructed recently, their performance to liquefaction and liquefaction-induced lateral spreading is expected to result in damage to the abutment and approach due to the lack of widely disseminated information regarding the assessment and mitigation of liquefaction-induced forces and displacements.

In addition to the comparisons between base shear coefficients of different design eras the changes in the design of bridge foundation, detailing seismic resisting elements for ductile response and the design of inter-span linkage systems was summarised for each design era. These comparisons in seismic detailing used with the ratios of base shear coefficients can be used to help identify potentially vulnerable bridges during a seismic screening of New Zealand road networks owned by local authorities or aide in detailed assessments of bridges that lack complete construction or design documentation.

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