Abstract:

In December 2004, Transit New Zealand issued a provisional amendment to the Bridge Manual introducing revised earthquake loading and concrete durability requirements. Dramatically different draft requirements proposed for the New Zealand Standards NZS 1170.5 and NZS 3101, undergoing revision but with finalisation having become protracted, had undermined the credibility of the previously existing Bridge Manual requirements.

Since then, both revised New Zealand Standards have been published, and a detailed review undertaken of the Bridge Manual seismic design requirements.

Significant aspects revised in the amendment include the following:
- Categorisation of bridges for importance in a manner similar to that for buildings set out in NZS 1170.0
- Derivation of design annual probabilities of exceedance for the ultimate limit state and for the serviceability limit states 1 and 2.
- Design philosophy for the design of bridges for earthquake resistance
- Derivation of the design earthquake loading. Included in this are criteria for developing site specific hazard spectra, and requirements for when these are mandatory.
- Methods for combining the responses in different orthogonal directions
- The effects of Liquefaction of site soils
- Dynamic analysis
- Seismic displacements and limitation of displacement
- Capacity design principles
- Design for serviceability limit state 2 (operational functionality)
- Requirements for tight linkages and span/support overlap

In developing this amendment, in addition to NZS 1170.5 and AS/NZS 1170.0, particular consideration was also given to the recommendations of the report MCEER/ATC-49, “Recommended LFRD Guidelines for the Seismic Design of Highway Bridges.”

Possible future directions for the ongoing development of the Bridge Manual are also briefly reviewed. Concrete and steel design standards have recently been revised or are under revision. A Land Transport NZ research project: “Review of AS 5100 Australian Bridge Design Code with a View to Adoption” has been concluded, and a number of Design & Construct projects have been tendered. A number of issues have been identified from these that need to be addressed.

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New Zealand Transport Agency Bridge Manual – Recent Amendment for Seismic Resistant Design, and Future Directions

D.K. Kirkcaldie, P. Brabhaharan and R Kotze

Introduction

Published initially in 2003, the New Zealand Transport Agency’s Bridge Manual, Second Edition\(^1\) aims to be a living document that is revised regularly to incorporate the latest thinking and practices in bridge design and to maintain alignment with the national and international design standards that it draws on.

Previous amendments to the Bridge Manual\(^1\) adopted in 2004 were reported at the 2006 Austroads Bridge Conference (Kirkcaldie, 2006\(^2\)) and the New Zealand Geotechnical Society Symposium (Brabhaharan, 2006\(^3\)). Since that time, two further topic areas of the Bridge Manual\(^1\), Earthquake Resistant Design and Concrete Durability, have been revised. These topic areas, had already been the subject of a Provisional Amendment issued in December 2004 as the finalisation of the corresponding New Zealand standards, NZS 1170.5:2004\(^4\) and NZS 3101:2006\(^5\), had become protracted and had undermined the credibility of the existing Bridge Manual\(^1\) requirements. Publication of these standards has now enabled finalisation of the amendments to the Bridge Manual\(^1\). Other recent relevant publications have also been taken into account, most notably the MCEER/ATC-49 report: Recommended LRFD Guidelines for the Seismic Design of Highway Bridges\(^6\). The background to these revisions is the principal subject of this paper, but this paper also attempts to look forward and to suggest possible areas in need of amendment in the near future.

Recent Amendments for Seismic Resistant Design

Categorisation of Bridges for Importance and Derivation of Design Annual Probabilities of Exceedance

AS/NZS 1170.0: 2002\(^7\) introduced the approach of categorising structures according to their importance, taking into account both their function and the consequences of their failure, as the basis for establishing the annual probability of occurrence of natural environmental events to be designed for. This approach had previously been incorporated into the Bridge Manual\(^1\) for the derivation of design wind, snow and floodwater actions, and has now been extended to include earthquake actions.

The Importance Levels (IL) assigned in Table 2.1 have been developed to reflect the consequences of failure of bridges. This includes (in general terms) the human risk that such a failure may have and aligns to a similar acceptably low probability of occurrence during the intended life of the structure as has been used in the assignment of importance classes to buildings given in tables 3.1 and 3.2 of AS/NZS 1170.0\(^7\). For bridges, however, the probable economic loss to the community that may result from a bridge failure is also considered when assigning importance levels. This is in recognition of the importance of the communication and traffic links that bridges fulfil and that their failure will have widespread impact on the community and its ability to recover from damaging earthquake. Such assessments remain subjective, but are expected to form a component of the design report, which should include the importance level assigned and some justification to underpin that assignment. It should be noted that the design levels for bridges of importance levels 3 and 4 are however the same, this being a reflection firstly of the...
subjective differentiation between ‘high risk’ and ‘very high risk’, and secondly that, once
the loading intensity exceeds a threshold, then the influence of ‘load intensity’ on in-service
performance is outweighed by other attributes of the construction such as workmanship,
unforeseen ground conditions and the like, in which cases the performance benefit gained
by increasing the load intensity is negligible.

The acceptable annual probability of exceedence (APE) values used in AS/NZS 1170.0(7)
have generally been retained as the basis for the values used in the table, with the
reference point being adjusted (to 1/1000 from 1/500) in recognition of the longer intended
life of bridges compared to buildings (i.e. to 100 years from 50 years).

Annual probabilities of exceedance have been added to Table 2.1 for the earthquake load
case. The APE value for IL1 temporary structures for wind, snow and flood has been
amended to 1/50 in line with AS/NZS 1170.0(7) Commentary clause 3.4. The ultimate limit
state APE value for Importance Level 1 permanent structures has been set at 1/500
(compared to 1/250 in AS/NZS 1170.0(7)) as unlike the building structures covered by
AS/NZS 1170.0(7), these structures will still generally be public structures which are
lifelines for communities (though small) that rely on them. A sufficient level of strength
must therefore be provided to give reasonable assurance against collapse and loss of
access under a major earthquake.

For bridges of IL1, the acceptable probability of exceedence remains 10% in the intended
life of the structure, but with that intended life extended to 100 years (c.f. 50 years for
buildings) the ULS APE for IL2 bridges becomes 1/1000 years. Similarly, for importance
level 3 bridges, the acceptable probability of exceedence within the intended life of the
structure of 5% remains (APE = 1/2500). This level of action was retained for IL4 bridges
(rather than it be extended to 2% as has been used with buildings) in recognition of the
difficulty in differentiating between these two importance levels and also because this
value already reflects a very low acceptable probability of exceedence.

The definitions of SLS 1 and SLS 2 have been interchanged to align with a recent
amendment to AS/NZS 1170.0(7) which interchanged the definitions of these serviceability
limit states.

While the principle of operational continuity for all traffic at SLS2 is now specified at a level
of ULS design of APE = 1/500, it is the linkage of ULS loads (e.g. APE = 1/2500 for IL3
and IL 4 bridges) to the ability to repair these bridges for the passage of emergency traffic
within 3 days that may require most thought and particular attention. Thus, while there
remains a ULS requirement for adequate strength and ductility with the bridge structure,
there now needs to be a means of damage control or repair implementation within a
relatively short time frame.

Categorisation of Retaining Walls and Derivation of Annual Probabilities of Exceedence for
Design

Retaining walls have now also been categorised for importance level for their design for
environmental events in a similar manner to and consistent with that for bridges. The
definition of the importance levels for the retaining walls now focuses on:
• importance of the route
• consequences of failure in terms of duration and extent of disruption to road access.
• consequences of failure to any adjacent property supported or protected by the wall.
Consideration of the resilience of the structure in terms of the extent and duration of disruption to access is consistent with the performance based approach (Brabhaharan, 2006\textsuperscript{(8)}), see Figure 1 (smaller the area, greater the resilience).

![Figure 1 Conceptual Definition of Resilience](image)

The recent changes allow a lower design life than 100 years to be adopted for:
- Walls of Importance Level 1, as agreed with the Road Controlling Authority (RCA)
- Walls of Importance Level 2, but not less than 50 years, as agreed with the RCA

The design life for a Importance Level 3 or 4 wall shall not be less than 100 years.

Walls associated with bridges are to be designed for the same annual probability of exceedence events as the bridge, as previously.

It is noted that previously the *Bridge Manual*\textsuperscript{(1)} specified different earthquake design levels for retaining walls than bridges, whereas with the recent amendment, if the wall has a similar level of importance as a bridge, it would be designed for a similar probability of exceedence.

**Design Philosophy, Earthquake Actions and Structural Systems**

Clause 5.1.1 has been revised to align with clause 2.1.3 of the *Bridge Manual*\textsuperscript{(1)} and with AS/NZS 1170.0\textsuperscript{(7)}, which, for the serviceability limit state, now sets out two performance levels
- SLS 1 under which no damage is to be sustained, and
- SLS 2 under which post-earthquake operational continuity is to be assured following the specified annual probability of exceedance event.

The result is that the seismic performance requirements are now more specific in terms of the expected performance. Furthermore, clause 5.6.11 requires the SLS 2 limit state to be confirmed and presents criteria for the determination of associated P-delta effects.

The *Bridge Manual*\textsuperscript{(1)} previously presented the expectation that a structure designed for the design earthquake event should not collapse under a much larger event, commonly interpreted as the “maximum credible event”, which in turn is commonly assumed to be an event of about 2500 years return period. This expectation has been discarded in favour of the approach outlined in the Commentary to *NZS 1170.5*\textsuperscript{(4)}, clause C2.1 for treatment of the ultimate limit state, which is consistent with the importance categorisations and associated annual probabilities of exceedance of design earthquake events adopted.

Clause 5.1.3 Earthquake Actions has been added as a new clause providing a broad overview of the process of derivation of the design loading and design approach to providing earthquake resistance, while in clause 5.1.4 Structural Action, the philosophical requirements from *NZS 1170.5*\textsuperscript{(4)} in respect to load paths and ability to withstand structural...
deformations have been incorporated. The previous material related to definition of structural ductility and classification of structures by their mode of behaviour has been relocated to 5.3.4 and juxtaposed with 5.3.5 which defines the structural ductility factors permissible for adoption in design based on the classification of mode of behaviour.

Design Earthquake Loading and Requirements for Site Specific Spectra

Ongoing research over many years has led to a greatly improved understanding of the magnitude and distribution of seismic hazard throughout New Zealand. Updated design seismic hazard are now presented in NZS 1170.5(4) and reflect the following nature of changes to design earthquake loadings:

- For areas north of Palmerston North, the design earthquake loadings are generally similar to previously (lower or similar in Auckland, Hamilton and Tauranga, higher in Napier and lower in Wanganui)

- South of Palmerston North, the design earthquake loadings increase quite significantly in many areas (e.g. Wellington, Greymouth, Queenstown, but not Christchurch or Dunedin). In some areas, the increases are to such an extent that will be very demanding on design and can be expected to result in much more substantial than previously used bridge substructures.

Bridge Manual(1) clause 5.3.1 describes the derivation of the design earthquake loadings in general terms while clauses 5.3.2 and 5.3.3 incorporate the NZS 1170.5(4) seismic hazard spectra specified in clause 3.1 for horizontal loading and 3.2 for vertical loading.

Zone factors less than 0.13 have been introduced in the knowledge that the associated risk factor, $R_u$, used for most bridges will be between 1.3 and 1.8. If applied to artificially inflated Z factors, this would have resulted in design values much greater than expected from seismic hazard assessments. By reducing the values of the zone factor below 0.13 for locations north of Auckland, yet retaining the requirement that the product $ZR_u$ be not less than 0.13, a reasonable correlation between the seismic hazard and the acceptable annual probability of recurrence has been achieved.

With vertical seismic loading now required to be explicitly considered, the “k” factor previously applied to dead load in the ULS load combination 3A has been deleted.

The December 2004 Provisional Amendment essentially banned the use of site specific hazard spectra to derive design loadings less than those specified by the draft of NZS 1170.5(4) standard at the time, drawing attention to the facts that:

- the Bridge Manual(1) did not provide for site specific hazard spectra to be used as a basis for reducing the specified design earthquake loading

- the spectra specified by the Standards NZ loading standard and the Bridge Manual(1) incorporate a lower bound placed on the zone factor (Z) designed to ensure that structures designed for strength and ductility achieve the objectives that:

  - after the design return period event, the bridge should remain useable by emergency traffic, though temporary repairs may be required, and
be repairable to full design capacity, also possess sufficient capacity to avoid collapse under the maximum credible earthquake event for the site.

Clause 5.2 has now relaxed the ban placed on site specific studies being used to lower the design loading by the Provisional Amendment, but incorporates specific guidance on the derivation of site specific spectra with appropriate bounds on the spectra derived. The derivation of site specific hazard spectra are to be fully documented as special studies, which can then be subjected to peer review if desired by the road authority. Site specific studies remain mandatory when a site is in close proximity to an active fault, as previously, but requires that they also be mandatory for bridges of high value, in view of the extent of variability from the code spectra that can quite validly arise.

Methods for Combining Responses in Different Orthogonal Directions

Methods for combining responses in different orthogonal directions have been based on the proposals of the MCEER / ATC-49 report\(^6\).

Two different methods by which structural responses to earthquake motions in different orthogonal directions may be combined, have been incorporated. These are:

- the Square Root of the Sum of the Squares (SRSS) method, and
- the 100% - 40% Combination rule.

Where response to vertical earthquake motions is also to be combined with response to horizontal motions, only the SRSS combination method is permitted.

As explained by the MCEER/ATC-49 report: “The SRSS rule is the most appropriate rule for combining the contribution of orthogonal, and uncorrelated, ground motion components to a single seismic force. The SRSS method is recommended particularly for seismic analysis including vertical ground motions (Button et al. 1999\(^9\)). Prior AASHTO seismic provisions were based on a 100% - 30% combination. It was decided to modify this and permit a 100% - 40% combination rule as an alternative to the SRSS rule. The 100% - 40% combination of forces provides results similar to the SRSS combination when the same response spectrum is used in two orthogonal directions (Clough and Penzien, 1993\(^10\)).” The Bridge Manual\(^1\), similarly to AASHTO, had previously specified the 100% - 30% rule.

Where the site is within 50 km of a major active fault, giving rise to the likelihood of concurrent horizontal and vertical earthquake motions, response to vertical motions is to be combined with the response to horizontal motions. Where the site is more than 50 km from a major active fault, response to vertical motions is to be treated as non-concurrent with response to horizontal motions, as was previously allowed.

Derivation of Peak Ground Acceleration

One feature of the recent amendment has been to link the derivation of the design acceleration and design velocity for the design of retaining walls to the peak ground acceleration determined from NZS 1170.5\(^4\). Previously the Bridge Manual\(^1\) formulae for design acceleration and velocity were not directly related to the peak ground acceleration. The new formulae for design acceleration and design velocity are given in Equations 1 and 2.
Design acceleration
\[ C_0 g = C_n(T=0) Z R_u S_p g \]  
Design velocity
\[ v_0 = 1.5 C_n(T=0) Z R_u S_p \]

where
- \( C_0 \) = design ground acceleration coefficient
- \( g \) = acceleration due to gravity
- \( v_0 \) = design ground velocity (m/s)
- \( C_n(T=0) \) = spectral shape factor at period \( T=0 \) determined from NZS 1170.5 Clause 3.1.2 and Table 3.1, taking the values in brackets from the table for the appropriate subsoil category
- \( Z \) = hazard factor determined from NZS 1170.5
- \( R_u \) = return period factor determined from relevant annual probability of exceedence
- \( S_p \) = structural performance factor

The use of the structural performance factor here is questionable, but has been retained until further research is undertaken as to the need for any factor in this case. The current NZSEE study group on retaining walls could provide some guidance to proceed further.

Liquefaction of Site Soils

The previous amendment of the Bridge Manual\(^{(1)}\) introduced explicitly the need to consider the potential for liquefaction of soils and the consequences to highway structures.

The recent amendment specifies the derivation of peak ground acceleration associated with earthquakes in a manner consistent with NZS 1170.5\(^{(4)}\), see Equation 1, and using a structural performance factor \((S_p)\) of 1. This recognises that the structural performance factor is not relevant to the assessment of liquefaction, and ensures that the assessment using internationally recognised procedures such as those presented by the Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils\(^{(11)}\) is not distorted by a factor unrelated to the peak ground acceleration from the earthquake.

Seismic Displacements

Clause 5.5.7 requires \(P-\Delta\) effects to be taken into consideration. They are a real action that can act on the structure to induce instability, and a sound rational basis for the previous exemptions from needing to consider \(P-\Delta\) effects has not been able to be determined, so these have been deleted.

Clause 5.5.8 Seismic Displacements has been amended to achieve consistency with the nomenclature of NZS 1170.5\(^{(4)}\) and with clause 5.3.7.

Based on the recommendations of the MCEER/ATC-49 report\(^{(6)}\), Clause 5.5.9 Limitations on Displacement has been extended to include a further limitation designed to avoid “ratcheting” of the structural displacement that could lead to a large residual displacement and possible instability.

Capacity Design Principles

The over-riding design principles on which all the materials design standards are based are appropriately presented in the relevant “loadings” standard. NZS 1170.5\(^{(4)}\) does not do...
this well for capacity design, and this was not previously presented in the Bridge Manual\(^{(1)}\) and so has been incorporated. These principles are:

(a) That elements of the structure intended to dissipate seismic energy through inelastic deformation be designed to possess a minimum dependable strength, and to maintain their structural integrity sufficient to develop the necessary ductility without undue loss of strength.

(b) That other elements and members of the structure, intended to remain elastic during earthquake response, be designed to withstand the forces induced in them through the inelastic deforming elements developing their “overstrength” capacity under response of the structure to strong earthquake motions.

The traditional approach of adopting a strength reduction factor $\varnothing = 1.0$ for design in a material or mode of action to resist the forces developed in the structure by yielding elements developing their overstrength capacity under earthquake response is not considered to be appropriate when a low strength reduction factor would be applied to that material or mode of action for normal design. Examples of materials which, due to their high variability or uncertain reliability, low strength reduction factors are adopted for normal design include soils and FRP materials. Modes of action for which low strength reduction factors are adopted include those where failure is of a brittle form or the performance of less certain reliability (e.g. friction grip bolting).

The amendment now requires that the capacity of elements and members designed to remain elastic, and of inelastically deforming elements for modes of action other than that of the intended inelastic action, shall satisfy the following relationship:

$$\text{Element and member forces mobilised by inelastic elements developing their “overstrength” capacities} \leq \frac{\varnothing S_n}{\varnothing_i}$$

Where

- $S_n$ = nominal strength of the elastically responding element or member at the ultimate limit state for the relevant action, as defined by the relevant materials design standard
- $\varnothing$ = strength reduction factor at the ultimate limit state for the relevant action of the elastically responding element or member, as defined by the relevant materials design standard
- $\varnothing_i$ = strength reduction factor at the ultimate limit state for the action of inelastic behaviour of the inelastically deforming elements, as defined by the relevant materials design standard

This approach for adjusting the design strength adopted for resisting overstrength actions meets the intent of the traditional approach but caters appropriately and consistently for all materials and modes of action.

**Requirements for Tight Linkages and Span Support Overlap**

Clause 5.7.2 Horizontal Linkage Systems, item (b) Tight Linkages has been extended to more specifically define the derivation of the force that tight linkage systems are to be designed for.
Item (d), Overlap Requirements, has been amended to incorporate the span/support overlap requirement proposed by the MCEER/ATC-49 report\(^{(6)}\). This formula is similar in form to that recommended by Priestley, Seible and Calvi in their widely accepted book *Seismic Design and Retrofit of Bridges*\(^{(12)}\), and allows for:

- relative displacement due to out-of-phase ground motion of the piers
- rotation of pier footings, and
- longitudinal and transverse deformation of the piers.

**Future Directions**

**Adequacy for Design & Construct Procurement Methods**

NZTA’s preferred procurement method for large projects in recent years has been Design & Construct contracts, or in the case of some very large projects, the Alliance approach. These forms of contracts provide a very strong driver to adopt minimum initial construction cost solutions, and require very comprehensive specification of the principal’s requirements in order to achieve the quality of product desired when considered in terms of level of service required at minimum whole-of-life cost and acceptable levels of performance in large low frequency events such as earthquakes.

In the light of several recent and current Alliance and Design & Construct contract experiences, a detailed review of the *Bridge Manual*\(^{(1)}\) is required to assess how well it is serving in meeting the demands of these forms of procurement. A number of topics flagged from recently tendered Design & Construct contracts, where upgrading of the *Bridge Manual*\(^{(1)}\) requirements is needed, include the following:

- D&C design reports and design approval
- Design loading for fatigue
- Design requirements for collision on bridges with structure above deck level
- Durability/corrosion protection requirements
- Design requirements for the detailing barriers
- Clearances required between barriers and structure behind
- Integral abutments and design for strain ratcheting of the backfill stiffness
- Requirements specific to network arch bridges
- Equestrian use of bridges
- Seismic design requirements for slopes and embankment fills
- The use of risk based approaches where the cost of *Bridge Manual*\(^{(1)}\) specified requirements to achieve an acceptable level of performance is excessive

**Harmonisation with Updated Materials Standards**

An updated edition of the *NZS 3101 Concrete Structures*\(^{(5)}\) was published in 2006 with two further amendments issued since. This standard highlighted a number of areas in which it does not provide sufficient coverage for bridge design. Also significant changes have been made to the sections related to prestressed concrete design and concrete durability in particular. While still in draft form, this standard was reviewed to identify implications for the Bridge Manual. Amendments to the *Bridge Manual*\(^{(1)}\) now need to be formulated and promulgated to achieve full harmonisation.

*NZS 3404 Steel Structures Standard*\(^{(13)}\) is currently undergoing revision with the draft of part 1 now available for public comment. NZTA needs to become an active participant in the development of this standard in order to ensure that the needs of bridge and roading...
structures engineering are adequately met. Again, amendments to the Bridge Manual\textsuperscript{(1)} can be expected to be needed in order to maintain harmony.

**Ongoing Harmonisation with Australian Practice**

The Land Transport New Zealand research project: *Review of Australian Standard AS 5100 Bridge Design with a View to Adoption*\textsuperscript{(14)} has now been completed and published as an NZTA research report. This report has flagged many areas where differences in practice arise, and while not recommending adoption of *AS 5100*\textsuperscript{(15)}, does recommend a major revision of the Bridge Manual\textsuperscript{(1)} to incorporate aspects of *AS 5100*\textsuperscript{(15)} that would enhance the present Bridge Manual\textsuperscript{(1)} provisions. Other overseas design standards also need to be monitored to identify new developments and these incorporated as appropriate.

**Structural Performance Factor $S_p$**

In the area of earthquake resistant design, debate continues on whether the structural performance factors specified by clause 5.3.6 of the Bridge Manual\textsuperscript{(1)} are appropriate. They differ from those specified by *NZS 1170.5*\textsuperscript{(4)} on the basis of a judgement that bridges will derive less damping from the influence of secondary elements than buildings and be more influenced by soil structure interaction, with softer soils providing more damping than stiff soils or rock. Also as noted above, the use of a $S_p$ factor in the derivation of design accelerations for retaining walls is questionable, and needs to be resolved. Formation of a study group to develop a consensus on this issue is recommended.

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**References**


