Seismic Assessment of State Highway Bridges in New Zealand – 20 years later

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D.R. Novakov & G. Adhikari

Opus International Consultants Ltd, Wellington.

G. Gregg

New Zealand Transport Agency, Wellington (retired).

ABSTRACT: In 1998 the New Zealand Transport Agency started a project of Seismic Screening, Assessment and Retrofit of bridges on State Highways in New Zealand. The project started with Stage 1: Seismic Screening, completed in 2002, followed by Stage 2: detailed seismic assessments and retrofits. Numerous challenges have been encountered during Stage 2. The paper discusses some of these challenges and how they have been dealt with, with the aim to assist both the owners of the bridges and the engineers who are involved in this type of work.

1 INTRODUCTION

In 1993, then Transit New Zealand and now New Zealand Transport Agency, commissioned a pilot project to develop a seismic screening procedure for state highway bridges, for which it is responsible. The project resulted in the publication of Transit Research Report Nº 58 (Transit, 1996), followed by the Manual for Seismic Screening of Bridges (Transit, 1998). In the same year Transit started an ambitious project for the Seismic Screening of bridges. This desk top study (Stage 1), completed in 2002, identified 335 bridges that required a detailed seismic assessment (DSA) and a further 170 bridges with inadequate or non-existent linkages between spans necessary to reduce the risk of span unseating. Based on the seismic hazard at the site and importance and vulnerability of the bridges, a priority list was created and the work on Stage 2 - DSA and seismic retrofit started in 2003. This paper discusses some of the challenges and how these were dealt with since the commencement of Stage 2.

2 SCOPE OF THE PROJECT AND CURRENT STATUS

With almost 11,000 km of state highways (and a further 83,000km of local roads) New Zealand is amongst the countries with the highest length of roads per capita population. It was considered prudent at the start of the project to categorise SH routes into three priority groups (Chapman et al, 2005): Priority 1 routes (with more than 4,000 vehicles per day that provide essential link to large centres of population or carry significant numbers of commercial vehicles), Priority 2 routes (carrying between 1,000 and 4,000 vehicles per day, or where the routes provide alternative access to large centres of population) and Priority 3 routes (the remaining low volume highways in the network that are largely intra-regional in character).

Seismic screening and the subsequent ranking of bridges (Stage 1) was completed using a range of structural and non-structural criteria. These are discussed in detail in Chapman (2005). Based on the results of the screening, completed bridges were separated and ranked into those that are eliminated from further assessment (as considered low risk or as on programme for replacement), those identified as having linkage deficiencies and those requiring a detailed assessment. About 170 bridges were found to require improvement to their inter-span linkages, of which 166 have been retrofitted as part of this project. Of the 335 bridges identified as requiring detailed assessment, 192 were on Priority route 1. To date more than 150 detailed seismic assessments of those have been completed and more than 40 bridges have been, or are in the process of being retrofitted as part of the project.

3 PROCESS – FROM DETAILED SEISMIC ASSESSMENT TO RETROFIT

In Stage 2 the process followed these four steps: 1 – Detailed Seismic Assessment (DSA), 2 – preparation of a Seismic Retrofit Design Statement (DS), 3 – design of the retrofit and preparation of Contract Documentation (CD), and 4 – construction. DSA reports, DS and CD were all subject to the scrutiny of NZ Transport Agency appointed peer reviewers.

4 ISSUES RELATED TO DETAILED SEISMIC ASSESSMENT AND RETROFIT DESIGN

In this section we discuss briefly the methodology and issues encountered during the assessment and the design of seismic retrofitting.

4.1 Methodology

The detailed seismic assessment of bridges was completed following the Direct Displacement Based Method (DDBM), using a non-linear push-over analysis by computer modelling and generally following the FEMA 440 model, with reference to Priestley (1996, 2007) and NZSEE (2006).

4.1.1 *Seismic performance criteria and retrofit design criteria*

New bridges on state highways in New Zealand are now designed (Bridge Manual, 2014) assuming a 100 year "design life" and a 2,500 year return period shaking as an Ultimate Limit State (ULS) design level. It is recognised, however, that if adopted for the seismic assessment of existing bridge stock which is typically more than 40 years old, the above criteria may lead to economically unjustifiable retrofits. Instead, the following standards (for assessment and retrofit) have been adopted: for bridges with a remaining life of 50 years or more, a 1,000 year return period (ULS) event and for those with a remaining life of less than 50 years, a 500 year return period (ULS) event. Where the retrofit options are expected to cost more than 40% of the bridge replacement value, a lesser standard may be agreed. The lower limits adopted were 150 and 250 year return period (ULS) events for bridges with less than 50 and more than 50 years remaining life respectively.

Further to the above, the level at which the margin against the collapse of the bridge becomes low (the Collapse Limit State (CLS) prevention event) should not be less than 500 years return period event. This is an absolute minimum but generally the CLS event was adopted as being equivalent to level of shaking 1.5 times that adopted for the ULS event.

4.1.2 *Seismic demand*

Seismic demand was derived following the recommendations of NZS 1170.5:2004 and as modified by the Bridge Manual. A further modification made in the DDBM assessment is that the Ductility Factor, k_μ, was set to unity. Instead of assumed ductility, the effects of energy dissipation were allowed for via the equivalent viscous damping in the system, assessed following the Modified Acceleration Displacement Response spectra method (MADRS) (FEMA, 440). Also, the Structural Performance Factor, S_p, was set to unity. This factor allows for some additional damping in the supporting soil, so using a lower value than unity would be a duplication of the effect when the MADRS method discussed above is used in the assessment.

4.1.3 *Material and section properties*

Probable, rather than the dependable material properties were used in the determination of the section and member capacities. This was done so that the actual, rather than the design capacity of the structure can be estimated. Effective "cracked" section properties were assumed for all concrete sections where member cracking / yielding is exceeded.

4.1.4 *Deck diaphragms*

Concrete decks have been considered typically as rigid diaphragms. Where linkages are provided across joints, their stiffness and strength has been considered in the modelling of the diaphragm action.

4.1.5 *Bridge skew*

The effect of the bridge skew was considered by 3D modelling and by applying seismic load in the relevant / critical load directions. It was agreed with NZ Transport Agency that, considering the complexity of this type of analysis and the Structural Performance factor, S_p of unity, the effect of the skew needs only to be investigated for bridges with a skew larger than 25º.

4.1.6 *Soil –structure interaction*

This was typically modelled via use of non-linear elastic perfectly plastic soil springs. Lower and upper bound soil properties were used to investigate the sensitivity of bridge response to these parameters. The force-deformation curve of each soil spring behind the abutments was modelled following the recommendations of either Caltrans (2013) or Khalili-Tehrani (2010). The ultimate resistance and secant stiffness to the yield displacement for pile soil springs were determined from the recommendations of Lam and Martin (1986). Ground conditions were assessed from geology and site reconnaissance to supplement any information on the drawings based on an understanding of past design and construction practices. Where important, these have been supplemented by site specific geotechnical investigations to determine the critical geotechnical issues.

4.1.7 *Section / Member capacities, ductility capacity and overstrength*

Member capacities were determined following relevant New Zealand and international material standards and with reference to Priestley et al (1996, 2007) and NZSEE (2006). Since the aim was to assess the probable, rather than the nominal (design) capacities, strength reduction factors (in reference to NZ standards) equal to unity were used. Where required, the post elastic (ductility) capacity was determined by limiting material strains, either in concrete or in reinforcing bars, depending on the expected failure modes, following FHWA (2006) guidelines.

Shear demand was assessed from the overstrength in the plastic hinges. Shear capacity within plastic hinges was assessed allowing for reduction due to the post elastic demand. The shear capacity of the beam column joints were also investigated to determine if plastic hinges in the adjoining members can develop.

4.2 **Issues related to seismic assessment**

Some of the issues encountered during the seismic assessment and how these were dealt with are presented as follows:

4.2.1 *Availability and quality of information*

Since the majority of the bridges were designed by the old Ministry of Works, or its predecessors (Public Works Department), original drawings were available for most of the bridges. In many cases these were "as-built" drawings. Some of the drawings contained information on the soil conditions at the site (e.g., borehole logs or description of soils), material properties (usually design concrete and reinforcing steel strengths) and a schedule of reinforcing.

The most common issues here were related to:

1. A lack of information relating to soils. In these cases, soil parameters were assessed from geological maps, observations made during the site visit and / or information obtained from projects completed on nearby sites, if available. A common issue was the lack of information needed to assess the susceptibility to liquefaction and effects of it, including the effects of lateral spreading. Where critical, geotechnical investigations and laboratory testing were used to assess the ground conditions and assess ground performance;
2. The degradation of strength due to aging e.g. corrosion of reinforcement, alkali silica reaction etc. State highway bridges in New Zealand are generally well maintained and in all but few cases it was assessed that the effect of the above are minimal, and;
3. Scour. In certain cases it was necessary to consider the effects of the observed or historically

recorded scour on the seismic response of a bridge. Many of the New Zealand rivers are braided and are also prone to sudden large changes in the volume and the speed of water within them. River beds consist typically of gravelly materials, resulting in significant scour around foundations, sometimes in the order of several metres. Ground levels on site, therefore, can vary from those shown in the drawings, affecting the stiffness of the bridge.

4.2.2 *Combination of bridge inertia loads with soil induced loads*

In addition to the inertial demands from the superstructure, the effects of surrounding soil movement were also taken into account. Where subsoil was susceptible to liquefaction under a design earthquake event, a displacement based approach was employed, in which the bridge was assessed for a set of ground displacement profiles representative of the ground movement during cyclic and liquefaction phases. To account for the fact that the peak inertia load and the peak kinematic load are not likely to occur concurrently, a portion of the inertia demand was combined with the full kinematic loading due to ground movement. Where the subsoil is not prone to liquefaction, only the soil inertia of the abutment backfill was considered.

4.2.3 *Travelling wave effect*

The effects of the "travelling wave" on longer bridges were analysed by assuming separate parts of the bridge responding in-phase or out-of-phase. Currently, only Eurocode provides some guidelines to account for this effect in an assessment. However, this was completed using a significant amount of engineering judgement and, hence the results may be subject to scrutiny.

4.2.4 *Modelling of expansion joints*

Most of the bridges analysed had one or more expansion joints. While the "design" width of the gaps in these joints were shown in the original drawings, the actual widths can differ, as observed on site for many reasons: creep and shrinkage in concrete, ambient temperature, movement of the supports etc. It was concluded that while the width of the gap assumed may have an effect on the sequence of events and the shape of the push over curve, it will not have a significant effect in the final conclusion regarding the seismic resistance of a bridge.

4.2.5 *Modelling and performance of hold down bolts*

Due to the detailing typical for NZ bridges, the hold down bolts under lateral loading are not likely to fail in simple shear. Instead, bolts are likely to deform within the oversized holes and also resist the load by tension. The capacity of the connection was assumed to occur either when the tensile capacity of the bolt or the local crushing of concrete around the hole edge occurs (Fischinger et al, 2013). This mode of failure are considered to be favourable to the brittle type shear failure.

4.2.6 *Modelling and performance of linkage systems*

Linkages typically consist of rods with nuts and rubber washers at one or both ends (therefore, mostly a "loose" type). Rods are installed through span end diaphragms and / or abutment back walls and / or pier shear keys. Capacity of the linkage system was typically low, limited by either: axial capacity of the bars, flexural or shear capacity of the end diaphragms and/or abutment shear walls. Low stiffness of the linkages (compared with the in-plane stiffness of the bridge deck) required explicit modelling of the former.

4.2.7 *Modeling of member Effective stiffness*

The relative stiffness of parts of the bridge determines where the inertia loads will migrate to. This stiffness changes during the response history of the structure (non-linear soil springs, plastic hinging). While the varying stiffness can be modelled, it makes the analysis more complex. Parts of the structure where plastic hinges are expected to occur were modelled using either distributed or lumped plasticity elements, while elsewhere an estimated constant value of effective stiffness was assumed.

4.2.8 *Assessment of section and member capacities*

Two major issues were encountered relating to the assessment of the section / member capacities, including their post-elastic (ductility) capacity. The first issue was the detailing of the existing bridge structure which, in many instances, did not comply with the requirements of the current design standards (inadequate reinforcement anchorage, lapping of bars within the plastic hinge regions, inadequate confinement, poorly detailed beam-column joints, lightly reinforced concrete sections and even total lack of reinforcing in members designed for predominantly gravity loads). The other issue is that the design standards are exactly that – the Design Standards – and as such are not necessarily suitable for the assessment purposes, as they are inherently conservative. It was therefore necessary to research various references, mainly the international standards related to assessment, and also use first principles and engineering judgement to assess the capacity of such members or sections.

4.2.9 *Consideration of shake off effects*

During the site inspections completed following the 2010 - 2011 Canterbury events, gaps between the piles, abutment retaining walls and / or pier walls and the surrounding soil were observed (Wood et al, 2012). These gaps, varying from a few millimetres to several centimetres wide, were an indication that the soil was irreversibly compressed by the movement of the bridge structure during seismic shaking. It was concluded that modelling of the shake off effect, while excessively complex, would be questionable, considering uncertainty in the soil properties assumed. The variation of the soil parameters (upper and lower bound) is judged to cover the above effects.

4.2.10 *Consideration of pounding effects*

The issue of the effects of pounding between parts of the bridge and how to model this is far from resolved. It was judged that although localised damage is likely to occur in most cases, pounding is not likely to cause significant damage and/or collapse of a bridge. On the other hand, pounding is likely to introduce additional damping in the system.

4.2.11 *Assessment of ULS versus CLS capacity*

In some instances it was not clear if a particular event (e.g. shear failure and / or reaching the ductility capacity of a pile) is an ULS or CLS event. Shear failure of an abutment pile within a stable embankment may not lead to a collapse, but a similar failure of the pier column could. Also, the ductility capacity of a plastic hinge limited by concrete cover spalling should not be considered as a CLS event, as a significant additional margin against collapse may be present (as long as adequate confinement to the main bars against buckling is provided). Unlike the above, collapse may be inevitable if the capacity is limited by the low cycle fatigue in an under-reinforced element, or where spalling of concrete at a section causes a significant reduction in gravity load carrying capacity. Therefore, the extent of the damage and its effect on the stability of the bridge were considered to determine if the member is at its ULS or CLS capacity. Inevitably, a substantial amount of engineering judgement is required to address the above.

4.2.12 *Liquefaction*

The assessment of the effect of liquefaction on the seismic response of a bridge is very complex. In most instances this was allowed for by a reduction in soil stiffness and capacity, by making an allowance for negative skin friction on piles, through the investigation of the effects of total or differential settlement of foundations and by applying additional loads on abutment foundations due to lateral spreading.

4.3 **Issues related to seismic retrofit design**

Some of the issues encountered during the seismic retrofit design and how these were dealt with are presented as follows:

4.3.1 *Inadequate ductility and or shear capacity of members*

Pier columns and other members above ground were typically strengthened by providing additional confinement to the potential plastic hinge regions or shear critical regions. Confinement was provided by Fibre Reinforced Polymer (FRP) wrapping or by the installation of steel or concrete jackets. However, for instances below ground, such retrofit was practical only where access and / or traffic management were not prohibitive. In many cases, the solution was to provide a retrofit that would reduce the demand on the critical elements or bypass them (via installation of new piles or sheet piling behind the abutments). When installing new jackets, it was necessary to ensure that this did not affect the flexural capacity of the member and shift the plastic hinge to the part of the member outside the retrofitted element.

4.3.2 *Inadequate capacity of beam-column joints*

Providing additional shear (splitting) capacity of the beam-column joints by FRP wrapping is typically not practical due to the convoluted shape of the joints (there could be as many as four beams and a column joining together). Instead the capacity was typically increased by the construction of reinforced concrete bolsters around the joint, or by additional post-tensioning of the joints.

4.3.3 *Inadequate capacity of hold down bolts and/or linkage systems*

This problem was resolved either by the installation of additional hold down bolts and / or linkages (sometimes requiring strengthening of the supporting elements), the installation of brackets and shear keys so as to bypass the inadequate hold down bolts, linkages, abutment back walls and deck end cross diaphragms, and / or the installation of steel "fish plates" connecting main steel girders to directly transfer deck inertia loads from one span to the other (bypassing the hold down bolts).

4.3.4 *Unseating*

In some cases, the demand on the existing hold-down bolts and linkages and their supporting element was just too big, the retrofit was not practical or would attract excessive loads onto the support. In such cases, to protect the support, connections were allowed to fail and seat extenders were installed to prevent unseating of the span(s).

4.3.5 *Steel truss bridges*

A typical weakness of this bridge type are the bearings (fixed sliding or rocker). Where the lateral capacity of these was inadequate, strengthening consisted of the installation of additional hold down bolts and steel stiffeners at the "fixed" bearings and shear keys at the sliding / rocking bearings. Also in some cases, where the truss was acting as a strut between piers and the leading abutment, the strengthening of the bottom chord members (normally loaded in tension) was required to prevent buckling. Strengthening of the cross bracing (typically designed for wind loading) was often required at bottom chord supported trussed spans where the deck is at the top chord level

4.3.6 *Steel girder bridges*

Steel girders integral (or not) with the concrete deck was a popular superstructure system in New Zealand, in particular during the 1960's and 1970's. While the girders could be quite deep, for larger spans, transverse diaphragms are typically very light and do not have adequate capacity to enable transfer of the inertia loads from the deck to the girder supports. Strengthening applied usually consists of the installation of additional transverse bracing (diaphragms).

4.3.7 *Dealing with more than one deficiency*

In some cases a global solution was considered as potentially cheaper than the retrofit of numerous separate deficiencies. On a particular bridge, the solution adopted was to install large diameter bored steel encased concrete piles behind the abutments. The effect was that these decreased lateral displacement of the bridge, reduced the load demand and / or post-elastic demand on critical elements and the likelihood of pounding and unseating. However, while the cost of installation of the new piles was lower than that of the alternative options (strengthening of individual members), the cost of traffic management and services relocations was very high, at almost half of the total retrofit cost.

4.3.8 *Finally - Liquefaction*

Retrofit to address the effects of liquefaction is very complex and usually expensive. It requires much more room than this paper allows. In most instances it has been accepted that the risk is there and that spending large amounts of money would be difficult to economically justify. In some cases, ground improvement using stone columns, buttresses or soil nailing was used to protect the bridge abutments.

5 A FEW EXAMPLES

A few examples of details encountered during the project and of retrofit solutions are presented below.

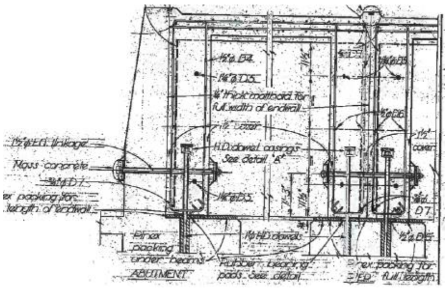

Fig. 1: Example detail of hold-down bolts and linkages

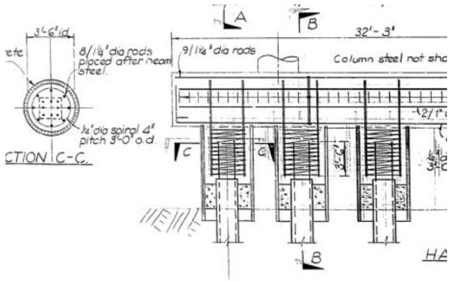

Fig. 2: Unusual detail of extended piles

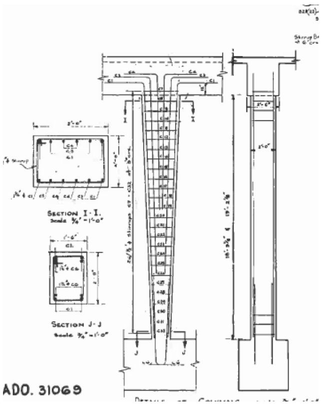

Fig. 3: Example of pier column (note main bars bending detail)

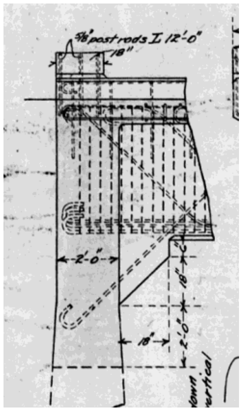

Fig. 4: Example of abutment detail of integral bridge

Fig. 5: FRP confinement of columns

Fig. 6: New shear keys and brackets

Fig. 7: New linkages

6 CONCLUSIONS

The paper presents an abridged history of the Seismic Retrofit of New Zealand State Highway Bridges project that has been going on for the last 20 or so years. Some of the issues and challenges encountered and solutions developed in due course are briefly discussed, together with the solutions for those. In summary, from our experience:

1. Seismic assessment requires a different mindset to that typically used in design. There is no room for excessive conservativism so to avoid unnecessary, expensive retrofits. Consequently existing design standards are not necessarily the best tools for this work.

2. It is paramount that realistic load paths are identified and critical weaknesses identified. There is no bigger nightmare for an engineer, and bigger loss for the community, than retrofitting of the wrong part of the bridge.

3. Knowledge related to the response of structures to seismic shaking is increasing on a daily basis. It is important to keep in touch with latest developments. Our methodology used in the assessment is constantly changing, from one bridge assessment to another.

4. Recent earthquakes (2010-11, Canterbury, 2016 Kumatoto/Kaikoura) provided a valuable opportunity to learn from the performance of bridges in real events. Bridges performed generally well (and better than predicted). However, they performed poorly in areas subject to liquefaction which highlights the importance to consider these effects. This is a challenge given the cost of investigations and also our current understanding the issues; and finally

5. There is no more fun for an engineer than doing this type of work.

7 ACKNOWLEDGMENTS

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And many more.