Displacement Based Seismic Assessment of Bridges in New Zealand

Dejan Novakov*, Robert Davey** & Donald Kirkcaldie***

Synopsis

A number of detailed seismic assessments of existing bridges has been completed for the New Zealand Transport Agency (NZTA). The majority of these bridges were designed and detailed pre 1975, to the standard much lower than that required for new bridges designed to the current NZ codes.

The “traditional” linear Force-based method (FBM) of assessment is simple, however, it has shortcomings which, while still acceptable for the new design, may not be desirable when assessing seismic performance of existing structures.

The Displacement-based method (DBM) combined with a non-linear push-over analysis, adopted in the project, starts from the premises that: a) internal actions (forces/stresses) in the structure are a consequence of the displacement rather than the cause of it, and b) strength capacity is less important than the displacement capacity. The inelastic displacement (ductility) capacity of an existing structure is, therefore, more important than its strength capacity.

The methodology of DBM used in the assessment of seismic performance of existing bridges is discussed and illustrated here on two examples.

Acknowledgments

The authors of this paper are grateful to the New Zealand Transport Agency (NZTA) for supporting and approving publication of this paper.

*Opus International Consultants Ltd, Wellington, New Zealand. Dejan is a structural engineer with 23 years experience of which over 16 years in New Zealand, specialising in site specific seismic hazard assessments and seismic assessment and retrofit of buildings, bridges and storage tanks.

**Opus International Consultants Ltd, Wellington, New Zealand), Manager Civil Engineering. Robert has over 30 years experience of structural engineering specialising in seismic hazard assessments and structural seismic assessments and design.

***Opus International Consultants Ltd, Wellington, New Zealand, Principal, Bridging and Earthquake Engineering. Donald leads Bridge Design team in Opus Wellington office. He has 30 years experience as a specialist in bridge engineering and was heavily involved in Transit's seismic screening of bridges project.
1 Introduction

A seismic screening of a large number of State Highway bridges in New Zealand was completed in about 2001. The purpose of the screening was to identify potential seismic deficiencies of these structures and rank them, according to their vulnerability and importance so that an order of priority for detailed seismic assessments and/or retrofit of these bridges can be established. The work on the detailed seismic assessments and retrofits has commenced in 2007 and is ongoing.

The majority of the screened bridges were designed pre 1975, when the first “modern” seismic design codes were introduced in NZ. The majority of the bridges assessed to date were designed and constructed in 1950-ies and 1960-ies, and, therefore, to seismic lateral loads much lower than that required for new bridges designed to the current NZ standards. The detailing of the bridges is typically inferior to the requirements of the current NZ material codes.

In this paper we commence with a brief discussion on not only the significance of the original design load but also on the detailing of these bridges, followed by the discussion of the assessment and retrofit design criteria adopted in the project. In our discussion on the methodology used in the assessment of the seismic capacity and performance of existing bridges we concentrate on the differences between the traditional Force-based method (FBM) and the Displacement-based method (DBM) of assessment without going into details of other steps which are common to both methods. Finally, we conclude this paper with the discussion on the benefits and limitations of the DBM of assessment and the importance of assumptions that are made during the seismic assessment process. We also provide, in the appendix, two examples to illustrate the assessment procedure used in the project.

2 Original Design Standards Compared Against Current Practice

Depending on their age, the assessed bridges are likely to have been designed to one of the following documents:

- Road Bridges – Loads and Allowable Stresses, Public Works Department, New Zealand, 1933
- Highway Bridge Design Loadings and Tentative Preliminary Code, Technical Memorandum Nº 8, Public Works Department, 1943, or
- NZ Bridge Manual 1956

All of the above documents prescribed bridges to be designed to resist horizontal seismic load of one-tenth the weight of the structure (i.e., in the current terms, for the design coefficient \( C = 0.1 \) g), independently of the location of a bridge, its importance, structural characteristics and/or the soil conditions at the site.

For comparison, the current Bridge Manual (2006) prescribes the following elastic (ductility \( \mu = 1 \)) design acceleration coefficients for new bridges on State Highways (\( R = 1.8 \)) with short structural periods, on Class C soil sites for some of the major centres:

- Auckland \( C = 0.43 \) g
- Wellington \( C = 1.36 \) g
- Christchurch \( C = 0.75 \) g
It could be expected, based on the above and considering only seismic strength, that the bridges designed to the above, pre 1975 standards, would have seismic capacity below that required from new bridges by a factor of many times. However, before any conclusion is made about the seismic strength of existing bridges the following should be considered:

a) The early standards were prepared based on the “Working Stress” method of design, while the current Standards/Bridge Manual is an Ultimate Limit State (ULS) standard. A commonly accepted conversion of the seismic coefficient from working stress to ULS is to multiply it by a factor of 1.2.

b) It can be expected that existing bridges have greater than the minimum seismic strength for various reasons, for example by rounding the number of reinforcing bars to a convenient number, by choosing the next size up of the steel profile, etc. Design for gravity (dead and live) loads will also commonly result in the bridges having greater than the minimum seismic strength.

c) Actual (probable) strength of construction materials is higher than the strength values (dependable) used in the design. Also, strength of concrete increases with time.

All these factors combined lead to a conclusion that the actual seismic strength of the assessed structures will typically be much higher than if only the original design coefficient is considered.

Further to the above, it has been recognised in recent times that the strength capacity is less important than the displacement capacity. Although the concept of ductility was not widely recognised at the time of the original design (certainly not in the loading standards of the day) and the existing structures would not have been designed and, more importantly, detailed for the post-elastic response, they do, inherently, possess some ductility capacity. As long as they do not fail in a brittle manner (e.g., in shear) or do not have some other major structural deficiency, it can be expected that existing structural members possess some post-elastic deformation capacity to, say ductility $\mu = 2$ and, in some cases (e.g., rocking piers, well confined pier columns and/or piles etc.), even more.

Ductility (i.e., ability of the structure to withstand limited post-elastic deformation), has two major affects on in-elastically responding structures:

a) Due to “softening” of the structure its natural period increases leading, to a reduction in the acceleration demand. The displacement demand, however, would remain about the same or could be higher, depending on the structural period, level of ductility demand and type of hysteretic behaviour of inelastic elements, and

b) Significant amount of seismic energy can be absorbed through hysteretic response of inelastic elements. This is presented via the amount of the equivalent viscous damping in the element (structure). Increase in the equivalent damping in the system leads to a decrease in seismic demand, both acceleration and displacement.

In the light of the above the seismic capacity of existing bridges might be higher than initially assumed. The DBM takes the above into account in the assessment of the seismic performance of existing bridges, as discussed in the following sections.
3 Seismic Assessment and Retrofit Design Criteria


The client’s brief required the design standard to “...be as specified in Transit’s Bridge Design Manual...”, but that "...where the cost of meeting the level of seismic code requirements are out of proportion to the benefits gained, the Consultant shall investigate lesser retrofit measures as he sees appropriate...”. The remaining design life of the bridge should be also considered when deciding on the design standard.

In view of the age of the assessed bridges and the precedent established by the previous retrofits of significant bridges of similar construction completed in the past, we have adopted a 1000 year return period earthquake intensity of shaking as the benchmark event against which the performance of the bridge has been assessed (the “design” event in terms of the Bridge Manual). This has also been adopted as the target minimum level of performance to be sought for the retrofitted bridge, together with collapse avoidance under the 2500 year return period event, in the expectation that full retrofit to achieve the 2500 year return period level of performance may not be justifiable.

4 Seismic Assessment of Existing Bridges

In the process of the seismic assessment of existing bridges we have followed these steps:

a) The assessment of the geology and the seismic hazard at the site.
b) The assessment of lateral load resisting systems (LLRS) of the bridge
c) The assessment of the soil parameters, susceptibility to liquefaction and slope instability.
d) The assessment of the material properties of the critical elements of the LLRS.
e) The assessment of the seismic capacity of the critical elements of the LLRS.
f) Mathematical modelling of the LLRS.
g) Computer analysis of the mathematical model.
h) The assessment of the seismic performance of the bridge and the seismic demand.

We skip here all of the above steps but the last two, as the former follow well established engineering practice. We provide a brief discussion on the type of computer analysis and concrete on the method of assessment of the seismic performance, as follows.

4.1 Method of Analysis of the Mathematical Model

A Non-linear Static (push-over) method of analysis was employed in the assessment of the seismic response of existing bridges. As discussed above it was expected that under the levels of seismic shaking defined above existing bridge structures would be pushed well beyond their elastic capacity. The non-linear method of analysis was,
therefore, considered more appropriate for this project. Non-linear methods allow “tracking of inelastic behaviour” of such structures and provide much more accurate assessment of their ultimate capacity.

It is recognised that higher mode effects are generally less significant for bridges than for building structures. Exceptions could be long bridges, with long spans and with supports of varying stiffness (i.e., irregular bridges), in which case their seismic response in transverse direction could be affected with higher modes of response. It is, however, likely that this effect would be more pronounced for superstructure transverse moments (typically not of great concern) and abutment reactions.

The results of a non-linear push-over analysis are presented by a graph of a base shear coefficient (expressed here as a seismic coefficient) versus displacement (of the centre of the superstructure mass). These results are then used in the assessment of the seismic performance of the bridge under the specified seismic demand.

4.2 Assessment of Seismic Performance

The traditional method of seismic assessment of existing structures is based on the Force-based method (FBM). The reason for this is mostly historical, as this method is normally used for structural design. More recently, as in this project, a Displacement-based method combined with non-linear static analysis are used. The two methods are discussed in the following sections.

a) Linear Force-based Method of Assessment

Linear FBM, when applied in the new design, is based on estimating structural dimensions and, from there, stiffness and natural period, determining seismic forces from elastic acceleration spectrum modified by the force reduction factor (as a function of the selected ductility), design and detailing of elements (following detailing requirements the selected ductility level) and, finally, checking of the displacement criteria.

When applied for the assessment of existing structures the method is slightly modified. It starts with the determining the probable stiffness and natural period of the structure and the assessment of the likely level of ductility available and, based on the latter, of the force reduction factor. Finally, strength capacity is assessed and compared with the strength demand and the assessed structure rated against the capacity of a new structure. If an assessment of the displacement criteria is made then either an equal displacement (for longer period structures) or equal energy (for short period structures) assumption is used.

There are, however, problems and/or deficiencies associated with this method, in particular when assessing the existing structures, as follows:

a) The method is based on assessed global ductility of the structural system i.e., on the assumption that a single force-reduction factor is appropriate for the whole structure. This assumption is invalid in particular for structures with dual (elastic and inelastic) load paths, a common system for bridges
where inelastic response is expected from piers but not from the superstructure (deck). For bridges with a continuous deck diaphragm seismic load in the transverse directions is shared initially (during elastic response) between the piers and the abutments, via the deck, according to the relative stiffness of the load paths. However, once the piers, which typically take most of the load, go into post-elastic range the load sharing alters significantly, with more load going into abutments, putting significant in-plane bending demand on the deck and its connections. While the force demand on piers is limited by its inelastic behaviour, the demand on the deck, considering that it responds elastically, and, therefore, on the abutments, increases continuously with the increase in displacement until the maximum displacement for the system is reached.

b) The method relies on estimates of initial stiffness to determine period and distribution of design forces between different structural elements. However, member stiffness changes with the increase in load (e.g., cracking of reinforced concrete) and with the increase in post-elastic deformation (crushing and spalling of concrete in plastic hinge regions). As members do not yield simultaneously, the assumed distribution of design forces between elements based on the initial stiffness is incorrect.

c) The method does not provide a full picture of the post-elastic response of the structure, its degrading stiffness (or, in some cases its temporary increase in stiffness, as illustrated in example B,) and/or strength and how it affects the overall response of the structure.

d) The ultimate displacement of the system is estimated in FBM, using either equal energy or equal displacement assumption. However, both of these are only approximations, which could lead to a significant misjudgement of both displacement and ductility demand on the structure and its elements.

While the design using FBM, combined with capacity design principles and adequate detailing, despite the problems discussed above, provides satisfactory, more or less conservative new designs, for better understanding of post-elastic response and seismic capacity of existing structures, the Displacement-based method (DBM), discussed in the following section, is more appropriate.

b) Displacement-based Method of Assessment

There are several methods of the DBM, depending on the stiffness characterisation used (e.g., initial pre-yield stiffness modified through iterations to achieve the target displacement, or secant stiffness to maximum displacement) and on how hysteretic energy dissipation is handled (e.g., those that use inelastic spectra and those that represent ductility and energy dissipation through equivalent viscous damping).

One of the methods is the Direct Displacement-based method (DDBM), which characterises structure to be assessed, by a single-degree-of-freedom (SDOF) representation of performance of a Multi-degree-of-freedom (MDOF) structure at peak displacement response. The two most often used procedures for this method are:

- The ATC 40 Displacement Modification Method, which uses empirically derived coefficients to modify response of elastically responding equivalent SDOF model of the structure, and
• FEMA 356 Capacity Spectrum Method, which uses empirically derived relationships for the effective period and damping –as a function of ductility – to estimate the response of an equivalent linear SDOF oscillator.

We have opted for the Capacity Spectrum method initially developed in ATC 40. In this method the displacement demand is determined from the intersection of the capacity spectrum, derived from the pushover (capacity or power) curve obtained in a non-linear push-over analysis of the structure, with a demand response spectrum, modified to account for hysteretic damping effects.

It has been observed that the results (displacement demand) obtained using ATC 40 and FEMA 356 methods vary, in some instances, by a significant amount. It was also subsequently confirmed, in research in which a comparison of the results obtained using the above DBM with the results of non-liner time-history analyses (considered to be the “correct” values) was made. It was found that ATC 40 method underestimates displacement demand for structures which undergo significant post-elastic demand i.e., structures with higher ductility demand (µ > 6.5), that is for structures with long effective (secant) period. To correct this, modifications to the ATC 40 formulas for equivalent damping and effective structural period. were proposed in FEMA 440. This revised method was adopted in this project.

The adopted DBM overcomes most of the deficiencies of the linear FBM discussed above, as follows:

a) The demand on the structure is determined from the actual in-elastic response of the whole structure instead from the assumed global ductility of the structural system
b) The method does not rely on the estimate of initial stiffness to determine period and distribution of design forces between different structural elements, as the change in the stiffness, and the distribution of internal forces is updated through each step of the push-over analysis.
c) The method provide better understanding of the response of the in-elastic structure, and
d) The displacement demand is assessed at the last step of the method when the capacity and the demand spectra intersect, which includes effects of stiffness degradation and energy absorption in the system. This enables an assessment of the ductility demand on all elements that undergo post-elastic deformation (e.g., yielding or rocking) and which may vary from member to member.

The ATC 40/FEMA 440 Capacity Spectrum method follows the following steps, with reference Figures 1 and 2:

1 A non-linear push-over analysis is performed. The result is presented in a form of a base shear – displacement curve, the so-called Capacity Curve (Figure 1). The capacity curve is then transformed into capacity spectra in an acceleration – displacement response spectra format (by scaling base shear and roof displacement by mass coefficient and mode participation factor for the first mode respectively) and plotted on a graph (Figure 2). Seismic response of most of the assessed bridges is dominated by the first mode of response (i.e., it is virtually a Single Degree of Freedom structure). Therefore, the modal mass coefficient
and the modal participation factors are equal to (or very close to) unity and the capacity spectrum is virtually identical to the capacity curve.

The 5 percent damped (elastic) Acceleration – Displacement demand spectrum is plotted on the same graph (Figure 2).

A trial (first iteration) performance point (a displacement-acceleration pair) the capacity spectrum is chosen. This must be point on the capacity spectrum in order to represent the structure and can be where capacity spectrum intersects the 5% damped demand spectrum, or at the displacement obtained using equal displacement approximation, or any other point chosen on the basis of engineering judgement.

A bilinear representation of the capacity spectrum (thin straight lines on Figure 2) is developed.

The effective structural period and equivalent damping are calculated using formulae provided.

A modified demand spectrum for the equivalent damping calculated in Step 5 is constructed and plotted on the same graph (dashed line).

The intersection point of the capacity spectrum and the modified demand spectrum developed in Step 6 above is determined.

If the intersection point determined in Step 7 above matches or is sufficiently close (within the acceptable tolerance) to the trial performance point chosen in step 3, then the trial performance point is the performance point and the displacement value of that point represents the maximum structural displacement expected for the demand earthquake. If, however, the intersection point is not within the acceptable tolerance then select a new trial performance point (usually the intersection point determined in the previous iteration) and return to Step 4.

**Figure 1:** Example of the Capacity Curve showing critical events obtained through a non-linear push-over analysis (note base shear is presented as a seismic coefficient)
Figure 2: Example of the DDBMA process - superposition of the Capacity Spectra (identical to the Capacity curve – see text) and the Demand Spectra with 5% (elastic) damping and % (equivalent viscous) damping.

The following can be observed from Figure 2 (similar applies to the figures in the Appendix):

- Yielding point in the equivalent system corresponds to spectral acceleration, \( Sa = 0.40g \), and spectral displacement, \( Sd = 18\text{mm} \).
- Displacement demand on the system is \( Sd = 41\text{mm} \) (1000 year return period).
- Ductility demand on the system is \( \mu = 2.3 \) (= 41 / 18) (1000 year return period).
- Estimated equivalent damping in the system is 10% of critical damping
- Estimated effective period of the system is \( Teff = 0.55\text{s} \)

Note that in the above example DBM predicts very similar displacement demand as it would be estimated using a linear Force-based method, combined with the equal displacement assumption. However, as the initial period of the example structure is about 0.42 seconds, it could be argued that an equal energy assumption should be employed, which would lead to a higher estimate of the displacement demand than by the DBM.

6 Examples

Two examples are provided in the Appendix to the paper. We comment briefly here on each of them.

a) Example A – Little Grey River Bridge

In the transverse direction the existing bridge has neither sufficient strength nor ductility capacity to satisfy the design demand. After the proposed retrofit however, it would have, as predicted by the DBM in Figure A3, sufficient strength to resist seismic shaking with a 1000 year return period, with a very small ductility demand.
(Sd (7% damping) = 0.8 g, Sd = 11 mm). For this system with predominantly elastic response a linear Force-based method would predict similar seismic performance as the DBM.

In the longitudinal direction the response of the structure is quite complex, with a lot of “lock in – unlock” events (Figure A4), making it almost impossible to reliably predict the performance of the bridge using linear analysis. A linear Force-based method (using initial period $T_0 = 1.12$ s, before abutment locks in – in this case incidentally matches the bold dashed line in Figure A5 which represents the secant stiffness of the in-elastic system) would overestimate displacement demand (obtained by extending the line until it intersects 5% damped spectra) as $S_d = 140$mm, compared with $S_d = 105$mm predicted in DBM.

b) Example B – Boundary Stream Bridge

In the transverse direction the existing bridge has very short initial period, leading to very high inertial forces, and a low strength capacity due to failure of the transverse diaphragms. Even retrofitted as proposed, the strength capacity of the bridge would still be low (Figure B2). Once the piers start rocking and the abutment hold down bolts fail the bridge would become an unrestrained rocking mechanism. An accurate assessment of the displacement demand is necessary for this structure in order to confirm if unseating of the girders at the abutments is likely, rather than to check the ductility demand in a “classical” sense. (e.g., checking post elastic rotation in plastic hinge regions)

It can be seen from Figure B3 that DBM predicts displacement demand of 275 mm. For comparison, a linear Force-based method would predict (using initial period $T_0 = 0.1$s and equal energy assumption) displacement demand much lower than that predicted by the DBM, leading to a non-conservative assessment of the seismic response of the bridge in this direction.

In the Longitudinal direction the response of the structure is characterised by a series of abrupt events creating a, so called, sawtooth curve (Figure B4). This indicates that the structural system of the bridge would change during a response to seismic shaking and, to use a term more appropriate for humans, is effectively a “triple personality” (Figures B5 and B6). Clearly, one would struggle to provide any reasonable answer using a linear Force-based method.

7 Conclusions

We have completed a number of detailed seismic assessments of seismic performance of existing bridges on State Highways in New Zealand. These bridges were designed to a much lower standard than the modern ones would be and their strength is, therefore expected to be low. It has been, however, recognised in recent times that strength capacity is less important than the displacement capacity.

In the assessment we have used a Displacement-based method of assessment which incorporates a non-linear static (push-over) analysis of mathematical (computer) models of the bridges. In this process a power curve (capacity spectrum), representing the in-elastic response of the structure to seismic shaking, is combined
with the acceleration-displacement demand spectra to enable an assessment of the seismic performance of the structure.

The DBM is an intuitive method which allows assessment of seismic performance of in-elasticly responding structures and overcomes many of the shortcomings of the more traditional Force-base method of assessment. It provides more believable assessment of performance than the Forced-based method with its assumed global ductility factor.

Several examples of competed assessments are presented in the appendix to this paper.

Results obtained through a DBM are, however, sensitive to the numerous assumptions that must be made during the assessment. Some of these are as follows:

- Choice of the load vector (load distribution within the structures) to model higher mode effects, if these are significant.
- An approximation of the capacity curve with a bi-linear curve to enable calculation of the ductility demand and equivalent damping in the system, and
- An assessment of the likely post-elastic behaviour of the structure i.e., of the critical inelastic elements (e.g., choice of the type of the hysteretic curve), which will determine the amount of the additional damping introduced into the system. At present different empirical equations are used to calculate equivalent damping and effective period of the structure, which appear to be calibrated towards the “right” answers obtained predominantly from the theoretical (computer model) analyses.

Displacement-based methods are still in the development and particular attention in the future research, in our opinion, needs to be put on addressing the last of the above issues.

A balance between the amount and/or variation of input parameters, complexity of the assessment process and the required accuracy of the results must be made when undertaking assessment of the likely seismic performance of existing structures. That is why seismic engineering is sometimes considered to be more an art than a science.

References

FEMA 440, 2005, Improvement of Nonlinear Static Assessment Procedures, Applied Technology Council (ATC 55 Project), State of California
Institute of Geological and Nuclear Sciences (GNS) web site (www.gns.cri.nz)

Appendix: Examples

The use of the Displacement-based method of seismic assessment is illustrated in several examples. The method was used not only for the assessment of existing structures but also to assess the seismic performance of these bridges if they were retrofitted as proposed in our assessment reports.

A Little Grey River Bridge

Little Grey River Bridge is a 10 span 196 m long bridge located on SH7 at RP 212/1.40, about 17 km south of Reefton. The bridge deck consists of a reinforced concrete slab supported on steel plate girders. Girders are continuous with only one in-span hinge at 6th span from Reefton abutment. Diaphragms are provided between steel girders at abutments and 1/3rd spans only. Expansion joints in the deck provided at the abutments and above the in-span hinge separate the superstructure into two rigid diaphragms. Both abutments are constructed of reinforced concrete and consist of two 1.2 m diameter piles joined with a deep cap beam. Bridge piers are constructed of reinforced concrete and consist of a wall supported by two 1.2 m diameter piles. In 2002 four pairs of new linkages were installed across the in-span hinge connecting each of the main girders. A new structural steel shear key was fitted at the same time, to the deck slab sofit across the deck joint above in-span hinge, keying against transverse differential movement. A sketch of the bridge is Figure A1 below.

The site is categorised as a Class C (Shallow soil) site, as defined in NZS 1170.5:2004. Zone factor is Z = 0.37.

The existing structure is assessed to be deficient to resist seismic loads imposed by a 1000 year return period shaking in the bridge transverse direction. The seismic performance of the bridge is currently limited by the lack of transverse diaphragms between the girders at the piers and the inadequate capacity of the existing diaphragms at the abutments. As a result lateral distortion of the girders is possible.
Expected performance of the bridge in the transverse direction is illustrated in Figure A2 below, which shows that the capacity spectrum does not intersect the 1000 year return period demand spectrum.

Figure A2 – Existing Bridge: Capacity spectrum versus demand spectrum for bridge response in transverse direction – upper bound of soil properties

A retrofit is proposed, which includes construction of new transverse diaphragms and installation of new shear keys at the piers. The expected seismic performance of the retrofitted bridge is illustrated in Figure A3. We have assessed that available ductility capacity of critical elements exceed ductility demand.

Figure A3 – Retrofitted Bridge: Capacity spectrum versus demand spectrum for bridge response in transverse direction – upper bound of soil properties

Seismic performance of the existing bridge in its longitudinal direction, as illustrated in Figures A4 and A5 below, is expected to be satisfactory. Ductility capacity of critical elements (pier walls and abutment back walls) was assessed to exceed ductility demand.
Although failure of the hold down bolts is expected during bridge response to seismic shaking well below the design level, unseating of the girders is unlikely, as the estimated displacement demand is much less than the seating provided.

B  Boundary Stream Bridge

Boundary Stream Bridge is located on SH 7, 14 km to the west from the intersection between SH 7 and SH 7A (leading to Hanmer Springs). The bridge is a two lane, three span structure approximately 83 m long. The bridge structure consists of reinforced concrete deck supported by simple span, steel plate girders. The reinforced concrete abutments are
supported on raking "H" section steel piles. The piers are supported on strip footings. Steel girders are connected to the abutments and piers by hold down bolts. In the longitudinal direction the girders are connected to each other and to the abutment back walls by linkage bars. In the transverse direction the girders are connected by steel trussed diaphragms at the supports and at 1/5 span locations (i.e., six diaphragms per span). A sketch of the bridge is Figure B1 below.

![Figure B1 – The Boundary Stream Bridge](image)

The site has been classified as site subsoil Class C (Shallow soil) site (NZS 1170.5:2004)

Six Class I active faults pass within 50 km of the Boundary Stream Bridge. The Hope Fault passes directly beneath the south-eastern end span of the bridge. A differential lateral movement of the two fault walls of between 2 and 5 metres is expected with a return period of 80 to 200 years. That amount of differential movement can, therefore, be expected between the two adjacent bridge supports, which could lead to the unseating of the superstructure.

The Zone Factor, Z, appropriate to the site is 0.55 (NZS 1170.5:2004).

The existing structure is assessed to be limited by the insufficient capacity of the transverse diaphragms between the girders, which are predicted to fail (buckle) at the level of seismic load with a return period of just over 100 years, resulting in possible excessive lateral distortion of the main girders.

A retrofit option is proposed which would include construction of new transverse diaphragms. Seismic performance of the retrofitted bridge in the transverse direction is illustrated in Figures B2 and B3 below. Although failure of the hold down bolts is expected during bridge response to seismic shaking well below the design level, making this bridge an unrestrained rocking mechanism, unseating of the girders is unlikely. Note that new auxiliary supports for the main girders have been proposed to address the issues related to the expected differential movement of the Hope Fault, however, the details of this are not discussed in this paper.
Seismic performance of the bridge in its longitudinal direction (Figures B4 to B6) below, is expected to be satisfactory. The “sawtooth” capacity curve shown in Figure B4 requires special care when determining the performance point. Curve # 2 intersects the 1000 year return period (design/performance level) at 120 mm spectral displacement, and Curve # 3 intersects the 2500 year return period (collapse prevention level) demand spectra at 350 mm spectral displacement. Unseating of the superstructure under these levels of displacement is not expected.
Figure B4 – Existing Bridge: Capacity spectrum for bridge response in longitudinal direction – lower bound of soil properties

Figure B5 – Existing Bridge: Capacity spectrum versus 1000 year return period demand spectrum for bridge response in longitudinal direction – upper bound of soil properties
Figure B6 – Existing Bridge: Capacity spectrum versus 2500 year return period demand spectrum (collapse prevention criteria) for bridge response in longitudinal direction – upper bound of soil properties