Seismic assessment of State Highway 16 Whau River Bridge

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ABSTRACT: The State Highway 16 (SH16) Causeway Upgrade Project consists of the widening and raising of 4.8 km of the existing SH16 motorway, and safeguarding it against future coastal erosion and flooding.

Two existing bridges on SH16, Whau River Bridge and Causeway Bridge, are being widened. Each bridge comprises two separate structures, built originally in the 1950s/1960s and widened in the 1990s. The bridges are a mix of structural forms and present many challenges for the integration of further widening work and for the assessment of the existing structural elements.

Comprehensive seismic assessments of the bridges in their proposed widened form have been undertaken. These have included performance-based nonlinear seismic evaluations following FEMA 440 improvements to the ATC40 methodology. This paper discusses the approach taken for the assessment of Whau River Bridge and how the results of the analyses have been used to estimate the functionality of the bridge under the design seismic event.

1 INTRODUCTION

1.1 Project details

The State Highway 16 (SH16) Causeway Upgrade Project is one of five packages comprising the Waterview Connection Project portfolio, which forms the final section of Auckland's Western Ring Route (WWR). The project is part of the NZ Transport Agency's Roads of National Significance programme, and when completed the WWR will form a vital alternative North-South route to the existing State Highway 1 for regional and national traffic movements in the Auckland area.

The Causeway Upgrade Project covers 4.8 km of the SH16 motorway from Great North Road Interchange in the east to the Whau River in the west. This section of the motorway is being widened, raised and safeguarded against future coastal erosion and flooding. One of the key features is the widening of the existing infrastructure to accommodate additional lanes carrying traffic from the adjacent State Highway 20 Waterview tunnel, due to open in early 2017.

The Causeway Upgrade Project is being delivered by the Causeway Alliance (an alliance of AECOM, Coffey, Fulton Hogan, Leighton Contractors, Sinclair Knight Merz and the NZ Transport Agency, who are also the client).

1.2 Existing bridges and proposed modifications

Two existing bridges on the SH16, Whau River Bridge and Causeway Bridge, are being widened as part of the project. Each structure was built originally in the 1950s/1960s and widened on its outer edge to provide additional carriageway width in the 1990s. The bridges are a mix of structural forms and present many challenges for the integration of further widening work and for the assessment of the existing structural elements.

Whau River Bridge is a 182 m-long, eight-span bridge crossing (Fig. 1a), comprising separate northern (eastbound) and southern (westbound) structures. It consists of reinforced concrete rectangular deck beams and deck slab cast monolithically onto reinforced concrete piled supports (Fig. 1b). The superstructure is fully integral with the substructure at all supports, including abutments, and the only articulation is provided by two full-width half-joints, one within the third span and one within the sixth span, which separate each structure into three sections of approximate length 60 m (Fig. 1c).

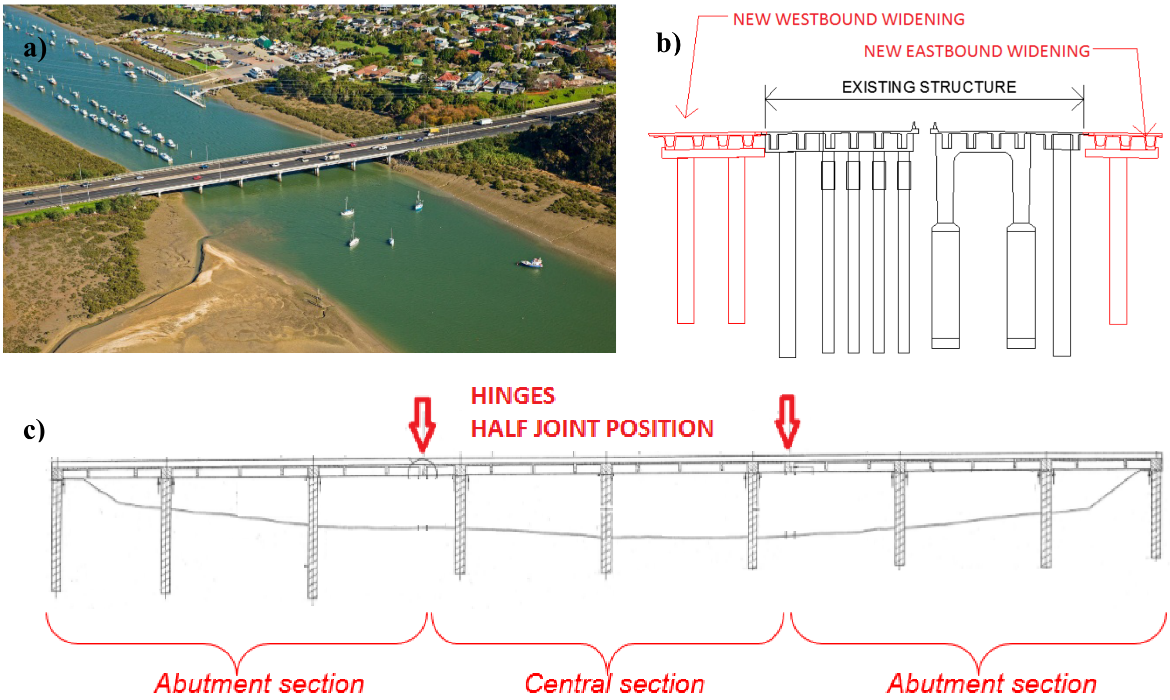

Figure 1. a) Aerial view of Whau River Bridge, looking south-west; b) Cross section of proposed widened bridge; c) Elevation along the bridge showing half-joint locations

The proposed widening works to the Whau River Bridge provide additional deck width on both outer edges (northern and southern). The additional deck width will accommodate a fourth traffic lane in both directions and a widened pedestrian/cycleway on the southern structure. The widening will be formed using precast pretensioned concrete 'super-T' beams and reinforced concrete deck slab, connected monolithically onto the top of new bored cast in-situ piles, and the new deck will be joined to the existing structure's outer edge via a reinforced concrete stitch pour. The existing half-joints will be extended across the new deck widening in order to match the current articulation.

The geology at the Whau River Bridge site generally comprises existing fill material and Tauranga Group marine alluvial material overlying weathered and unweathered Waitemata Group sandstones/ mudstones. Alluvial layers are relatively thin (2 m-3 m) near the west bank and thicken eastwards to around 10 m thick at the east end. Bedrock depth from ground level varies between 5 m and 15 m. The risk of liquefaction at the site was assessed to be negligible. Ground improvement works are provided at the eastern approach to the bridge to ensure that embankments are stable and that ground displacements do not affect the bridge foundations.

1.3 Seismic assessment requirements

The NZ Transport Agency's requirements for the project included the assessment of the bridge in its modified form against 'desirable' and 'minimum' seismic performance criteria. These criteria defined the acceptable damage and expected conditions of the bridge when subjected to an earthquake with a return period of 1 in 1000 years and to an earthquake with a return period of 1 in 2500 years. Table 1 below presents the criteria defined in the client's requirements.

Table 1. 'Desirable' and 'minimum' seismic performance criteria

	Structure	Criteria	1 in 1000 year event	1 in 2500 year event
Desirable	Existing and new widening	Level of damage	Minor damage	Moderate damage repairable to original capacity
		Level of service	No loss of service	Usable by emergency traffic and repairable to full service
Minimum	Existing	Level of damage	Moderate damage repairable to original capacity	No collapse
		Level of service	Usable by emergency traffic and repairable to full service	Complete loss of service
	New widening	Level of damage	Minor damage	Moderate damage repairable to original capacity
		Level of service	No loss of service	Usable by emergency traffic and repairable to full service

Each modified structure was to be initially assessed for structural adequacy under the desirable criteria. If found to be deficient, the structure was also to be assessed to confirm compliance against the minimum criteria, and the nature, extent and cost of strengthening work required to bring the structure up to the desirable criteria standard was to be determined so that the client could then decide, on a cost-against-risk basis whether such strengthening work should proceed.

2 ANALYSIS METHODOLOGY

2.1 General approach

A seismic assessment should be capable of tracing the expected force-deformation response of the structure, considering deterioration and damage to structural elements in order to evaluate safety (Priestley et al. 1996). A Force-Based Method (FBM) is usually appropriate for new structures design; however it presents shortcomings when assessing existing structures (Novakov et al. 2009). The most significant shortcomings of a FBM include:

1. Assumption of a global ductility for the structure: For bridges in particular, this assumption is invalid because of the generally elastic response of the deck in comparison with the inelastic response of the piers. The inelastic demand and ductility capacity of the piers are a function of the direction of analysis and the length of piles. Hence, the assumption of a constant force-reduction factor as a function of a global ductility capacity is inappropriate;
2. Computation of the fundamental period of the structure: In order to compute the seismic demand on a structure, a FBM requires the determination of the fundamental period, which is computed based upon assumed reduced section stiffness of the structural elements (cracked sections). The variability and inaccuracy of the assumption of cracked sections may significantly affect the demand forces. More importantly, it is also erroneously assumed that stiffness is independent of the strength (important considering that strength is only defined at the end of the design process). However, it has been proven (Priestley et al. 2007) that stiffness is essentially proportional to strength. Hence, it is not possible to perform an accurate analysis/assessment without considering the strength of the structural elements when computing the fundamental period of the structure;
3. Equal displacement assumption: the relationship between structural displacement and structural or non-structural damage is well established. An adequate estimation of the expected displacement demand of a structure is fundamental in order to estimate damage to it. A FBM assumes equal displacement, meaning that the inelastic peak displacement is equal to the elastic peak displacement, independent of the selected yield strength of the system. This assumption is correct in some cases; however it may lead to significant misjudgements of displacement demand, thus miscalculation of the expected damage to a structure.

Therefore, in order to have a better understanding of the seismic behaviour of the bridge, it was decided to use a displacement-based approach. A Displacement-Based Method (DBM), combined with non-linear pushover analyses, allows tracking of the inelastic behaviour of the structure rather than using an assumed global ductility. Furthermore, the method includes the effects of structural degradation and energy absorption, providing a more accurate evaluation of the displacement demands and the internal actions (forces/stresses) in the structure that are a consequence of the displacement demands; hence, the damage of a structure is better estimated using a DBM.

The inelastic seismic response of the bridge was investigated for both the 1 in 1000 year event and the 1 in 2500 year event. The likely 'collapse' mode/event was also investigated. The methodology consisted of comparing demand and capacity curves of the bridge following a DBM process specified in ATC40 (ATC 1996) and modified by FEMA 440 (FEMA 2005). The seismic demand was derived from NZS1170.5 (Standards New Zealand 2004). The capacity was assessed through non-linear static pushover analyses. For this, each structure (northern and southern) was separately modelled both two-dimensionally and three-dimensionally using a nonlinear analysis software package (SAP2000) and subjected to non-linear static pushover analyses in the longitudinal and transverse directions separately. The applied process can be summarized by the following steps:

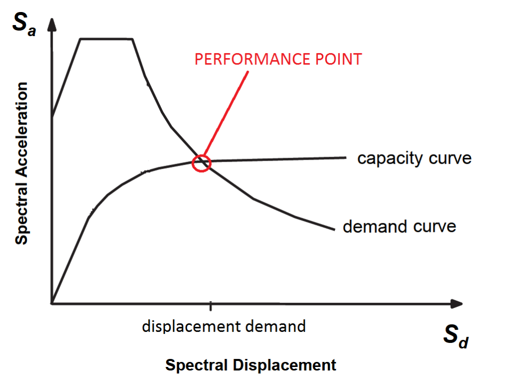

Figure 2. Estimation of displacement demand by intersecting capacity curve and displacement demand curve (performance point)

1. Determine the lateral resisting system and load paths;
2. Establish the potential collapse mechanisms;
3. Assess local weaknesses and their impact on the global inelastic behaviour;
4. Calculate the available ductility and deformation capacity of the structural elements (moment-curvature and section analysis);
5. Generate a numerical model incorporating inelastic behaviour of structural elements;
6. Perform pushover analyses of the bridge in the longitudinal and transverse directions. Obtain the capacity curve of the structure (force vs. lateral displacement);
7. Use a DBM to calculate the displacement demand of the bridge under defined return period events. The displacement spectrum is obtained from NZS 1170.5 initially assuming an equivalent viscous damping of 5%; this value is required to be updated based upon the determined ductility demand of the bridge, hence iteration is required;
8. Scale the capacity curve to transform it into spectral acceleration vs. spectral displacement format. For this, base shear is divided by weight coefficient and displacement by the mode participation factor. For structures where seismic response is clearly dominated by the first mode of vibration, such as the bridge under assessment, base shear is simply normalized by the seismic weight of the structure and the displacement remains invariable;
9. Plot on the same graph capacity curve and displacement demand (Fig. 2); the intersection of the curves (the 'performance point') provides the inputs to perform the first iteration of the displacement demand. From the displacement demand, an equivalent period, equivalent stiffness and base shear are obtained (Priestley et al. 2007);
10. Subject the numerical model to the computed displacement demand, and compute ductility demand for the lateral resistant structural elements with the formulae provided (Priestley et al. 2007). Obtain an updated value for the equivalent viscous damping;
11. Return to step 7 and repeat the process using the updated value of equivalent viscous damping until the displacement demand converges;
12. Apply the final displacement demand to the structure, and obtain force and displacement demand for the structural elements. Estimate damage to the structure.

2.2 Estimation of the expected damage

Performance requirements given by the client were related to allowable material strains in order to estimate structural damage (Table 2). It was considered that the creation of reversing plastic hinges is possible; hence the strain limits proposed were taken as 60% of those corresponding to unidirectional plastic hinges limits (Fenwick et al. 2007).

From the assessment process summarised above, it was possible to apply the displacement demand to the structure and obtain moment and shear demands. Main structural flexural, shear and axial capacities were then compared against corresponding seismic demand. Combining moment demand with section analysis (moment-curvature), it was possible to obtain the material strain demand and therefore the material condition and the expected level of damage.

Table 2. Performance criteria related to allowable material strains

Level of damage	Level of service	Allowable material strain		Material condition	
		Concrete	Steel	Concrete	Steel
Minor damage	No loss of service	0.0025	0.01	Cracking evident, but no significant spalling	Yielding evident, but no residual deformations expected
Moderate damage repairable to original capacity	Usable by emergency traffic and repairable to full service	0.0036	0.024	Spalling evident, but no significant reduction of strength	Yielding evident, and moderate residual deformations expected
No collapse	Complete loss of service	0.006(*)	0.054	Significant loss of strength, but section has not reached ultimate strain	Yielding evident and noticeable residual deformations expected. Ultimate strain has not been reached.

(*) A limit of 0.004 was used for concrete having inadequate confinement

The following damage definitions were used:

- Minor damage: Damage is manageable and repair costs should be economically feasible. No structural elements should fail. The bridge remains 100% operable with no loss of service;
- Moderate damage: Moderate structural damage may occur, but partial or total structural collapse is avoided. Some hinged columns are expected, but without generating any partial or total mechanism. It is possible to repair the structure. The bridge can be used by emergency traffic and is repairable to full service;
- No collapse: Substantial damage, such as multiple hinged columns, is expected, including significant degradation in the stiffness and strength reduction. Large permanent lateral deformation could occur, and live load carrying capacity is reduced significantly. Collapse is avoided; however, the bridge has a complete loss of service. Bridge replacement will be required due to high repair costs.

3 ASSESSMENT OF WHAU RIVER BRIDGE

3.1 Key assessment parameters

During the pushover analysis, only permanent dead loads were combined with seismic loads. Seismic loads were calculated in accordance with NZS 1170.5 (Standards New Zealand 2004) and the NZTA Bridge Manual Second Edition (NZ Transport Agency 2003) using the key values shown in Table 3.

Table 3. Summary of the key seismic parameters

Hazard factor (Z)	0.10 (*)
Site subsoil class	Class C soil
Importance level	3
Annual probability of exceedence:	Existing bridge: Desirable 1/2500, Minimum 1/1000 New bridge widening: 1/2500
Return period factor (Ru)	Existing bridge: Desirable 1.8, Minimum 1.3 New bridge widening 1.8
Structural performance factor (Sp)	0.8
Minimum horizontal seismic base shear force	0.05Wd [Wd: Seismic weight]

(*) ZRu not taken as less than 0.13

3.2 Numerical model

Whau River Bridge comprises, in total, six independent structural sections (three sections separated by half-joints on each of the northern and southern structures). Each structural section was separately modelled using SAP2000 and subjected to a non-linear static pushover and time history analysis in the longitudinal and transverse directions separately. Figure 3 shows the widened eastbound central section of the bridge subjected to a pushover. Additionally, each structure (northern and southern) was modelled as a whole, in order to evaluate possible pounding forces at half-joint locations.

Plastic hinges based upon axial and moment demand, as well as shear capacities of the key structural elements, were included in the numerical model in order to estimate possible mechanisms of failure.

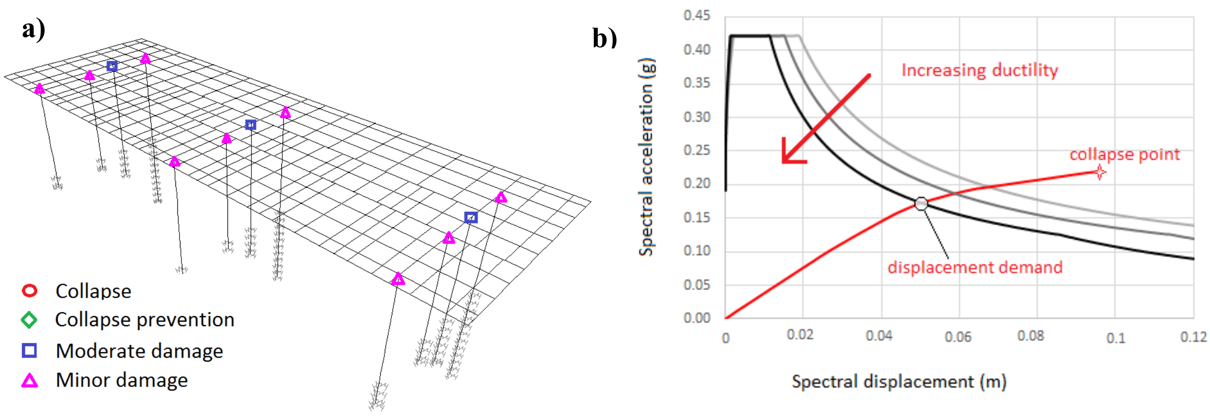

Figure 3. a) Longitudinal pushover view of the widened eastbound central section of the bridge showing expected level of damage to the piles; b) Estimation of displacement demand (demand curve shows iteration when increasing the equivalent viscous damping)

For the pushover analysis in the longitudinal direction of the bridge, the model was pushed at the centre of mass, i.e. the forces/displacements were applied only to one joint for each of the three longitudinal sections of the bridge. However, for the transverse pushover multiple points of pushing were simultaneously used. These points corresponded to each of the transverse frames at the piers and abutments of the bridge. Hence, the bridge was subjected to lateral forces distributed proportionally over the span of the bridge in accordance with the product of mass and mode shape. The bridge was pushed up to a target displacement (expected displacement demand) and the hinge formations of the bridge at different steps of the pushover procedure in the longitudinal and transverse direction were obtained.

To account for soil-structure interaction, the numerical model included linear springs on foundation members. A sensitivity analysis of the stiffness provided by the soil was also included to allow for upper and lower bound soil stiffness values. Additionally, spring capacity (compression) was limited to the maximum soil capacity. After the soil capacity had been reached, the soil was considered to carry that constant capacity, redistributing the additional load.

Structural member capacities were computed based upon ‘Assessment and improvement of structural performance of buildings in earthquakes’ (NZSEE 2006). Shear degradation of structural elements was considered based upon section curvature ductility (Priestley et al. 1996). Confined concrete models (Mander et al. 1988) were used to compute compressive stress-strain curves for piles.

3.3 **Critical weaknesses**

The original Whau River Bridge was designed prior to modern design codes. Hence, the level of detailing of the structure, in particular for connections, does not satisfy current seismic design requirements. Two principal deficient details were found to be present in the original structure:

1. The existing ’half-joint’ type joints (Fig. 4a) provide longitudinal movement capability for the deck under thermal expansion and contraction of the concrete. However, the joint detail may generate high pounding loads between deck sections during seismic movements. Half-joint seating is also of concern: the seating is nominally 150 mm wide based upon record drawings and could be reduced to 100 mm of effective seating during seismic displacement. Hence, the original design has used linkage bolts between deck sections in order to limit their relative movement and avoid unseating and local collapse of a deck section. The new widening sections have been provided with a similar half-joint detail with linkage bolts; however, seating width has also been increased;
2. The lack of confinement reinforcement in existing bridge portal frame knee joints (Fig. 4b) may lead to a brittle mode of collapse. The lack of stirrups in the joint and the poor anchorage detail of the column bars generate a weak point for which the mode of failure may be shear dominated.

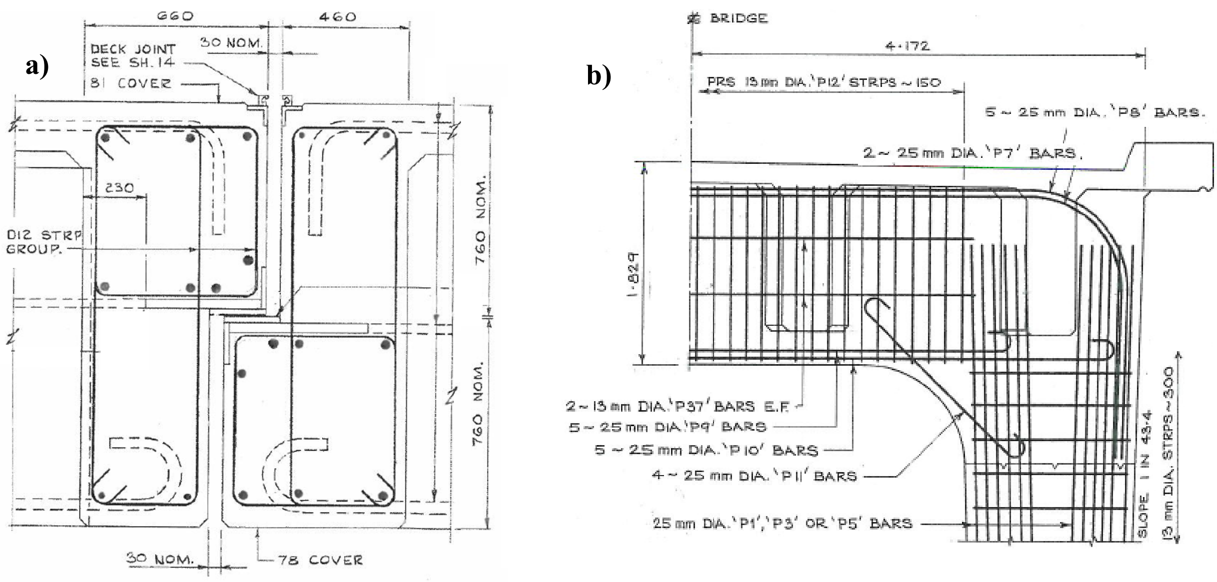

Figure 4. a) Existing half-joint sectional detail b) Existing unconfined portal frame knee joint detail

3.4 **Pounding at half joints**

For two adjacent sections of the deck (abutment and central sections) moving out-of-phase in a design seismic event, the total anticipated longitudinal displacement is 108 mm (76 mm movement of the central section and 32 mm movement of the abutment section). The maximum gap thickness at the half-joint is 50 mm, hence pounding between the two sections of the bridge is expected. An assessment of the pounding forces was therefore performed.

Time history analyses were carried out in order to compute the estimated pounding forces on the deck. A group of fifteen seismic records were selected and scaled to represent seismic conditions for a 1 in 2500 year return period earthquake. The scaling process was performed according to NZS 1170.5. Time history analyses were performed for each of the seismic records using SAP2000. Pounding was implemented using a contact force-based model (Des Roches et al. 2003). The results showed that the expected pounding force is similar in magnitude to the base seismic shear forces. Two possible failure modes due to pounding forces were examined:

1. Excessive bending force on piles creating plastic hinges and eventually a mechanism of collapse: this mechanism of collapse appeared not possible due to the limitation of the maximum longitudinal displacement of the bridge. Pushover analyses showed that a minimum displacement of 74 mm would be required to reach collapse; however the displacement is limited by the joint gap thickness to a maximum of 50 mm;
2. Excessive shear force on piles producing a brittle mode of failure: the expected maximum pounding forces are considerably smaller than the shear capacity provided by the group of piles for each deck section. Hence, a pile shear failure is not expected.

4 **EXPECTED SEISMIC PERFORMANCE**

The expected seismic performance of the proposed widened Whau River Bridge was obtained and compared to the design criteria. Overall, the structural performance of the eastbound and westbound structures was found to be similar.

For the design 1 in 1000 year event, it is expected that the bridge will perform well under seismic loading. 'Minor' damage only is likely to happen. Expected damage is manageable and repair costs should be economically feasible. The bridge would remain fully operable with no loss of service. Main structural elements may experience small repairable damage, and the deck may experience localised cracking due to pounding forces at the half-joints. No structural elements would fail.

It is expected that the bridge will perform well under the design 1 in 2500 year seismic loading. The longitudinal assessment indicated that 'minor' to 'moderate' structural damage may occur, but partial or total structural collapse is considered highly unlikely. It would be possible to repair the structure. The expected level of damage would allow the bridge to be used by emergency traffic immediately following the seismic event and the bridge would be repairable to full service. Piles are expected to experience moderate cracking and deck main structural members are expected to sustain minor structural damage; however, higher localized cracking is possible near the half-joints. Non-structural elements, such as asphalt, may suffer considerable cracking near the movement joints. The transverse assessment showed that 'minor' structural damage may occur, but that partial or total structural collapse would be avoided. Some hinged columns are expected, but without generating any partial or total mechanism. Lateral deck displacement measured at central piers in the order of 76 mm is expected.

In the longitudinal direction, two possible mechanisms of collapse are expected at seismic loading higher than the design events. The first case would be a shear failure of piles, and the second a loss of seating at half-joints. The shear failure of piles would control over a ductile mode of failure (for example, piles yielding due to bending producing a mechanism of collapse). The sudden loss of seating at half-joints (a brittle mode of failure) may occur as a consequence of a combination of high pounding forces causing shear failure at half-joints and vertical accelerations increasing vertical load at the joints. In the transverse direction a brittle mode of failure is expected as a consequence of excessive shear demand at the existing portal frame knee joint. Where a premature failure of the knee joint is controlled, a more desirable ductile mode of collapse could be achieved. However, knee joint failure is expected to happen for seismic forces around 25% larger than the seismic demand for a 1 in 2500 year event.

For the 1 in 1000 year seismic event the maximum transverse displacement expected is 54 mm at the central pier. For the 1 in 2500 year seismic event a value of 76mm of transverse displacement is expected, which is equivalent to 0.76% of drift. Hence the maximum displacement expected is around 20% of the recommended limits (Priestley et al. 1996). This level of displacement would not present any problems for pounding between the separate northern and southern structures at the carriageway median. In the longitudinal direction, the displacement is limited by the combination of half-joint gap thickness and resistance of abutment backfill. However, the expected maximum displacements of each section of the bridge were obtained by analysing without limitation for gap thickness. For the 1 in 1000 year seismic event the estimated maximum displacement of each abutment section is 23 mm and of the central section is 45 mm. For the 1 in 2500 year seismic event the maximum displacement of each abutment section is 32 mm and of the central section is 63 mm.

5 CONCLUSIONS

A performance-based seismic assessment of the proposed upgrade for Whau River Bridge was undertaken to estimate its seismic capacity and the possible failure modes under seismic loading. The inelastic seismic response of the bridge for both the 1 in 1000 year event and the 1 in 2500 year event was investigated. A suitable analysis approach was defined, linking specified structural damage and serviceability limits to allowable material strains and displacements. Due consideration was given to potential structural weaknesses under seismic loading, including pounding at half-joints and lack of confinement of existing portal frame knee joints.

It was found that under both seismic events the overall structural performance of the bridge is satisfactory. Expected damage is manageable and repair costs should be economically feasible. The bridge is likely to remain fully operable with no loss of service under a 1 in 1000 year event. For a 1 in 2500 year event, the expected level of damage would allow the bridge to be used immediately by emergency traffic in the aftermath of the event and repairable to full service. The controlling mode of failure is brittle, hence undesirable. However, the first mode of failure is expected to occur for loads around 25% greater than the 1in 2500 year event demand.

Computed pounding forces are not expected to produce collapse of the structure for the examined possible modes of failure. However, given the amount of uncertainty and the high values of pounding forces, it is possible that during a high seismic event the bridge (in particular the deck) may experience damage. The deck may experience localised cracking at half-joints due to pounding forces; however no structural elements would fail.

The assessment of the bridge using a DBM approach has shown that the design seismic criteria are able to be accommodated without the need for any structural modification or strengthening specifically for seismic performance. Earlier assessments using a FBM approach indicated that strengthening works would be required at certain substructure locations; hence, the use of the DBM approach has resulted in programming and cost savings for the project and a more suitable design outcome for the bridge widening works.

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