AUCKLAND HARBOUR BRIDGE SEISMIC RETROFIT

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SUMMARY

Following a detailed seismic assessment, the Auckland Harbour Bridge and approach structures are undergoing a seismic retrofit. Bridge components are being retrofitted to the adopted performance standard of immediate access to 4 lanes of traffic after a 2000 year return period event and a low risk of collapse during an MCE (maximum credible earthquake) event.

The bridge has a 244 m main span and comprises a steel truss bridge flanked on each side by steel box girder extension bridges. The retrofit work includes the following major items:

- Strengthening of the 'clip-on' steel extension bridge support brackets.
- Installation of braced steel frames and associated concrete works at the navigation span piers.
- Strengthening of the truss bridge deck bracing to protect against deck panel collapse.
- Installation of restraint bars to the steel box columns supporting the extension bridges, to protect against local buckling failure of the flanges.

INTRODUCTION

The 1.6km long Auckland Harbour Bridge is a key lifeline structure carrying eight traffic lanes, two water mains and other services between Auckland City and North Shore City. The original bridge, opened in 1959, is a four lane steel truss structure supported by cellular reinforced concrete piers sunk into sandstone. The bridge was widened during the late 1960's using two lane steel box girder structures supported by steel box trestles on steel box brackets stressed on to each side of the original concrete piers with prestressing bars.

The seismic assessment and retrofit design for the bridge has been carried out in four stages:

1. A preliminary assessment of the main bridge, approach structures, foundations and approach embankments to broadly identify potential vulnerabilities under seismic loading. This included a site specific seismic hazard analysis and the development of the required performance standard and assessment methodology (refer to [Billings and Kennedy, 1996], [Billings *et al,* 1996] for further details).

2. A detailed assessment to investigate the vulnerable components, including non-linear time history analyses using a combined model of the three bridge structures. The investigation scope was widened to cover assessment of the extension bridge support brackets and trestles for wind and live loading after deficiencies in these components were identified during the seismic assessment.

3. Conceptual design of retrofit solutions for vulnerable components, construction cost estimates and a probabilistic cost-benefit analysis.

4. Final design and contract documentation for the retrofit works.

This paper summarises the detailed assessment results and the retrofit design. The adopted performance standard for the retrofit design is summarised in Table 1. Components not satisfying these criteria prior to retrofit are referred to as "vulnerabilities."

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Table 1 Seismic Performance Standard for the Auckland Harbour Bridge

Ground Motion considered	Performance Required	Assessment Basis
Objective 1: 200 year return period motion	• Minimal damage • Immediate service to eight lanes	• Design strength • Serviceability
Objective 2: 2,000 year return period motion	• Low risk of loss of life • Repairable damage • Immediate access to four traffic lanes • Fully re-opened to traffic in a few days	• Nominal strength • Limit demand/capacity
Objective 3: MCE at mean plus 1.5 standard deviations.	• Low risk of collapse and major loss of life • Closure for an extended period is acceptable	• Probable strength • No collapse

DETAILED ASSESSMENT

In broad terms, the seismic assessment involved: assessing the material properties and condition of components, analysing the structure to find the seismic forces and displacements, assessing the strength and ductility of components, assessing the performance of critical components and the overall bridge performance against the chosen performance standard. Component assessment drew heavily on methods developed for major bridges in California since the 1989 Loma Prieta earthquake, particularly the work of Professor Astaneh of UC Berkeley and Professor Priestley of UC San Diego who have provided advice and reviews throughout the project.

Most of the seismic assessment work used the results of several 3D elastic spectral modal analyses. These runs used a site specific MCE spectrum (the Objective 3 ground motion as per Table 1). Additional runs for the 2000 and 200 year return period events were not needed, as scaling the MCE results provided sufficient accuracy.

Features of the three dimensional computer models (SAP 2000) used for the detailed assessment included:

- 3d frame model of the truss and extension bridge structures with stick models of the concrete piers.
- Simplified models of the steel pier extension brackets using properties derived from 3D FE models.
- Truss deck bracing represented by linear or secant stiffness springs derived from substructure models.
- Gap elements linking the truss bridge and extension bridge decks to model pounding and force transfer.

Several analyses runs were included in the member force envelopes to allow for combinations of the following:

- CQC modal combinations for 3 components of ground motion combined by the SRSS rule, or “30%” rule.
- Separated (open gaps) or linked (closed gaps) bridge decks in the linear analyses.
- Upper or lower bound stiffness assumptions for the truss deck model.
- Non-linear time history runs accounting for open/closed gaps. Records included El Centro 1940 NS (2/3 scale) and a spectrum matched synthetic record developed for the Auckland Sky Tower project.
- Static runs with dead loads, and relative pier displacements to represent differential ground movement.

Component checks for most truss bridge members were carried out using purpose written spreadsheets comparing conservatively calculated capacities to conservative combinations of member forces. Members failing these tests were scrutinised in more detail. Plate stress checks for the extension bridge box girders compared the gravity and seismic demands to those derived for gravity and wind loads in previous investigations. Potentially critical areas were then checked against design code limits.

During the seismic assessment of the extension bridge support brackets and trestles, potentially serious deficiencies under gravity loads were identified and the project workscope was widened to include an assessment of these components for all loadings. Analyses of these components included 3D linear finite element (FE) modelling of the brackets and 3D non-linear FE modelling of the diaphragm beams carrying vertical loads from the trestle legs into the bracket walls. Further field inspections were carried out to check plate and weld sizes, including ultrasonic testing to check root penetrations and integrity of critical fillet welds in the extension brackets. These investigations identified vulnerabilities under severe gravity and wind loading, the most critical components being the Pier 1 & 2 diaphragm beams. The 3D FE models indicated that yielding of these members was likely to have occurred during previous severe wind events. Close inspections of the diaphragm beams confirmed that yielding of the web plate and stiffeners had occurred on at least one occasion. The most critical components were immediately strengthened in 1997 as Stage 1 of the retrofit project

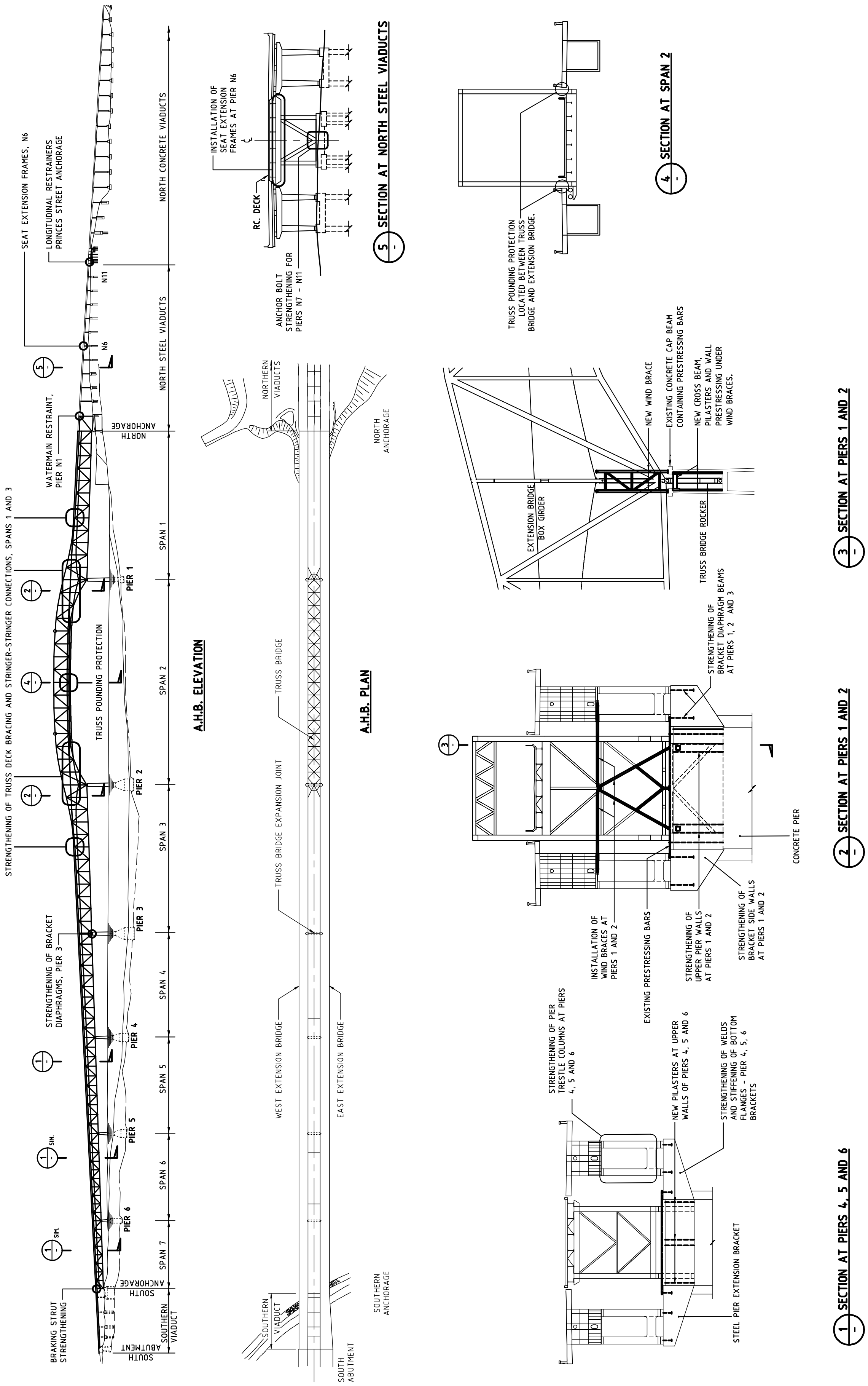

Figure 1 – Summary of Auckland Harbour Bridge Retrofits

MAJOR VULNERABILITIES AND RETROFIT SOLUTIONS

The major vulnerabilities identified during the seismic assessment are summarised in Table 2 along with the corresponding retrofit solutions. The locations of the components to be retrofitted are shown by Figure 1. The assessment findings and retrofit solutions for the four key retrofit items are discussed in further detail below.

Truss Bridge Deck Panel Bracing

Selected bracing components are being strengthened to avoid a major vulnerability in Spans 1 and 3 of the Truss Bridge where a combination of longitudinal loads, transverse loads and pounding between the adjacent bridge decks could cause sufficient damage for deck panel collapses to occur. The component failures required for this scenario to occur are:

- Multiple bracing member failures in spans 1 and 3 due primarily to transverse deck shear forces.
- Pounding damage at the centre of spans 1 and 3 causing failures in stringer bracing and fixing bolt fracture.
- Stringer–stringer seating failures due to subsequent longitudinal overloads (if bracing members have failed).

Once those components have failed, there is no longitudinal restraint to the truss posts at Piers 1 and 2 and the adjacent panels. A span collapse then becomes the likely outcome (but with a very low overall probability).

Extension Bridge Support Brackets

Several deficiencies were identified in the brackets supporting the Extension Bridges, as indicated in Table 2. Most are related to insufficient shear capacity in the fillet welds connecting the bracket side walls to the flanges and the diaphragm beams below the trestle columns. Other deficiencies were identified in the buckling resistance or yield strength of various plate elements including the diaphragm beams at piers 1, 2 and 3.

Evaluation of the of the diaphragm beams at piers 1 and 2 by hand calculation methods had indicated deficiencies in both bearing (crushing) resistance and in buckling resistance for seismic, gravity and wind loading. These calculations were later verified by inelastic post-buckling analysis using a detailed 3D FE model of the diaphragm beam (Fig. 2) which showed the collapse mechanism to be: a) yielding of the web and stiffeners under the load point creates a "pin", b) overall buckling of the "unstiffened" web plate follows. Inspections of the diaphragm beams indicated minor deformations consistent with this predicted behaviour.

The pier 1 and 2 bracket diaphragm beams have been retrofitted by fitting pairs of 200UC stiffeners to the web plates (Fig. 3) in 3 stages: a) temporarily bolting in position to secure against buckling, b) welding to the web plates outside the likely tension field zone, then c) fitting stub columns to bear against the underside of the bracket top plate below the trestle columns. The Pier 3 diaphragm beams require the first stage only.

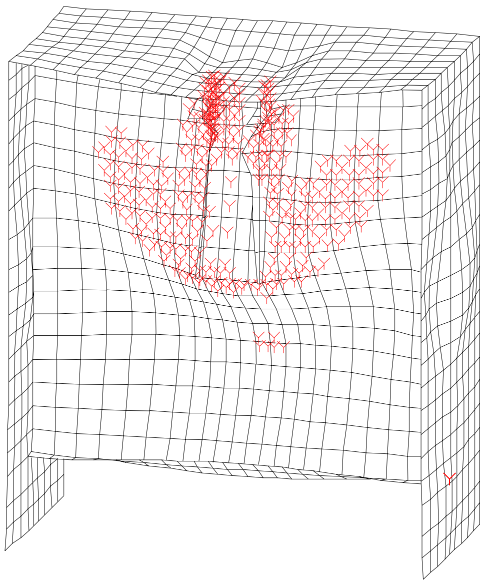

Figure 2 - Pier 1 & 2 Diaphragm Beam FEA

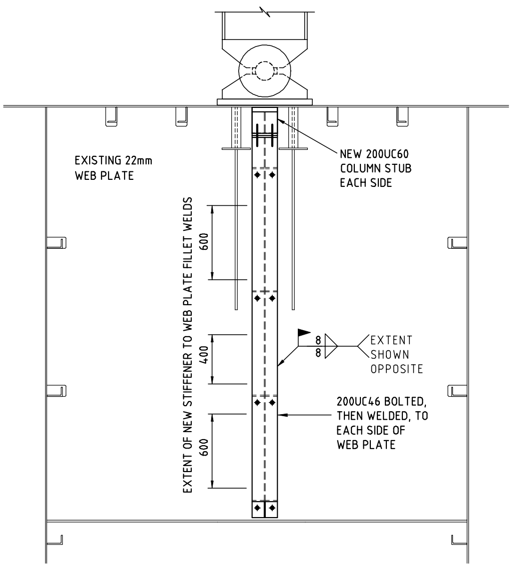

Figure 3 - Pier 1 & 2 Diaphragm Beam Retrofit

Deficiencies in weld shear capacities are being addressed by one of two methods. At the side-wall to bottom plate connection where the existing weld is a single side fillet, the risk of damaging the already highly stressed weld was such that the reinforcing angle detail shown in Fig. 4 was used instead of simply adding a fillet weld to the other side of the joint. At the diaphragm beam to side wall connections in piers 4, 5 and 6, the existing welds are to be reinforced by careful overwelding to increase the weld strength so as to exceed the diaphragm beam web shear strength.

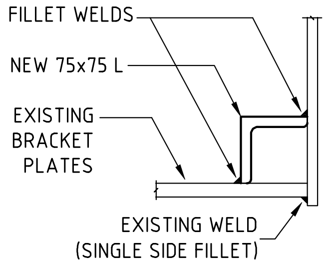

Figure 4 – Weld Retrofit

Extension Bridge Support Trestles

The Extension Bridge support trestles are steel box columns with a box beam above the lower rocker bearings. Plate slenderness ratios do not comply with modern seismic design standards and the web to flange plate welds are partial penetration butt welds. The butt welds connecting the upper rocker bearings to the underside of the box girders or the welds between the adjacent internal diaphragm and box girder flange plate do not have sufficient resistance to withstand MCE seismic actions.

At Piers 4, 5 and 6, local buckling of the column flange plates is expected at the MCE event loading. Although the assessed demand to capacity ratios were considered to be within acceptable limits [Astaneh, 1997], the partial penetration welds are considered to be at risk of "unzipping" following *any* local buckling deformation that causes joint "opening" forces. Furthermore, the occurrence of weld cracking adjacent to the inner edge of the upper rocker bearing can result in significant redistribution of load to one column, further increasing the risk of local buckling.

A variety of solutions were considered for addressing the local buckling and potential weld "unzipping" problems. Options such as weld repair and additional vertical welds to reinforcing plates were discarded due to potential problems with weld shrinkage stresses and long closures. The adopted solution is to connect the compression flange to the tension flange using restraint bars fitted through holes and welded to both flanges

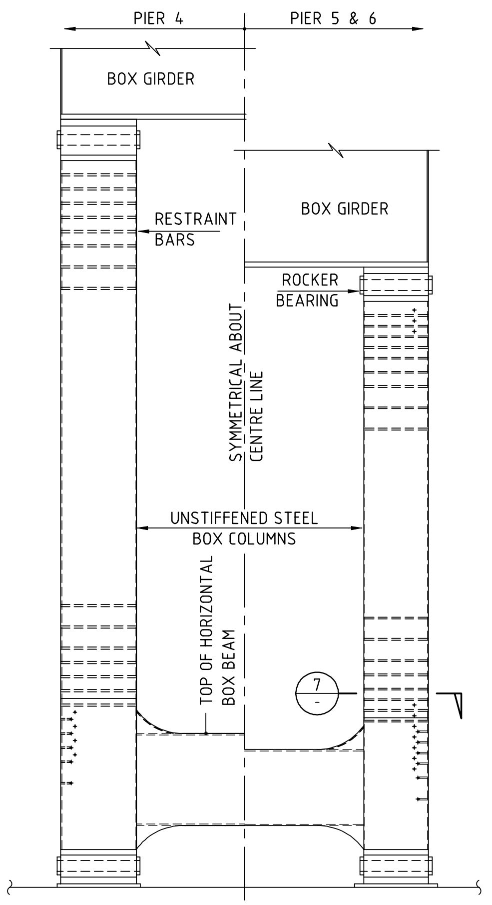

Figure 6 – Extension Trestle Column Retrofit

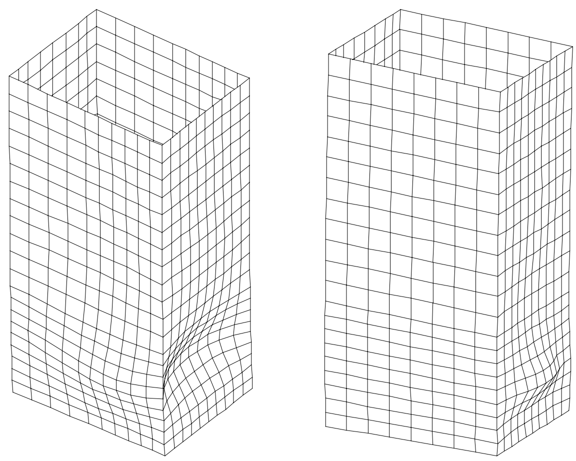

Figure 5 – Trestle Column FE Model, Before & After Adding Restraint Bars

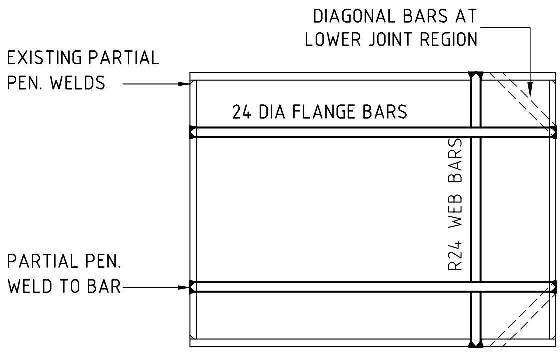

Figure 7 – Restraint Bars Fitted to Extension Trestle Columns

(Figs. 6 and 7). The distribution of forces and moments in the trestles is such that the tension flanges of the columns will remain elastic (and thus able to support face loads) except at plastic hinge rotations well beyond the demands predicted using the maximum displacements from the dynamic analyses.

The required spacings of the restraint bars were refined using inelastic post-buckling finite element models of the box columns (Fig. 5). These analyses indicated that local web buckling could occur in combination with local flange buckling, which would defeat the purpose of the flange restraint bars. To delay the onset of web buckling it was necessary to add more restraint bars between the web plates. For the lower portion of the trestle columns at the cross beam level, diagonal corner restraint bars are used instead of flange-to-flange bars.

The assessed performance of this retrofit solution is that significant opening actions at the column web to flange weld are avoided at the expected maximum plastic hinge rotations. Without the retrofit, both flange and web buckling would occur at lower hinge rotations, and weld "unzipping" would be unavoidable.

"Wind Brace" Concept for Piers 1 and 2

The solution adopted to address several vulnerabilities in the Extension Bridge supports at Piers 1 and 2 was to provide an alternative lateral load path from the extension bridge box girders to the concrete piers (Fig, 1). As the name "Wind Braces" indicates, their primary purpose is to support wind loads but they also address the Pier 1 and 2 trestle seismic vulnerabilities, and reduce the extent of the seismic retrofits required in the brackets. Further dynamic analyses including the wind braces did not indicate any detrimental effect on seismic response, other than an increased risk of pounding at the centre of Span 2 which is being addressed by adding local strengthening to the affected truss members.

The "inverted-V" braced frames are stressed onto the tops of the two upper walls of the piers using high strength bars anchored in the base slab below the truss bridge rockers through 11m long cored holes. Pairs of struts connect the apex of the braced frames to the extension bridges via connection plates bolted to the underside of the box girders, with pinned bearings used to accommodate longitudinal movements.

ECONOMIC ANALYSIS AND RETROFIT CONSTRUCTION

Major roading construction projects in New Zealand are required to have a cost-benefit analysis carried out so that funding priorities can be determined. A probabilistic economic analysis of the retrofit project was carried out using numerical simulation incorporating the seismic hazard model, traffic models, loading probability distributions, variations in damage and strengths of critical components, and various injury/fatality scenarios. This showed the expected Benefit/Cost ratio to be several times greater than the 4.0 required for project funding.

Retrofits of the most critical live load and wind load deficiencies in the extension pier brackets, including the Pier 1 and 2 diaphragm beam stiffeners, were completed in 1997. The second stage (completion of all retrofit measures described) is currently under construction, at a cost of approximately NZ$2.0M.

ACKNOWLEDGEMENTS

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Table 2 Major Vulnerabilities and Retrofit Solutions

Description of Vulnerability, Potential Consequences	Nature of damage to critical components	Vulnerability		Retrofit Solution
		Seismic	LL/WL	
Loss of support to individual truss bridge deck panels in spans 1 through 3 Collapse of deck panels, loss of central four traffic lanes. Possibly more global collapse of truss bridge due to damage to truss from falling deck panels. Severing of water mains, gas line, and other utilities.	• Failure of braking struts restraining deck panels in vicinity of Piers 1 and 2.	❍		Replace selected struts with larger size Tee sections and upgrade connections
	• Fracture of bolts in stringer-stringer and stringer-floor beam conn's due to impact loads. Net section fracture of stringer bracing connections.	❍		Replace stringer transverse braces, reinforce to avoid net section fracture. Replace critical bolts using long bolts with ductile sleeves.
	• Compression/flexure buckling of truss verticals adjacent to piers 1 and 2 following loss of support from deck panels	❍		Fit restrainer bars across sliding seat supports or upgrade the existing cleat connections, to maintain longitudinal load path.
Extension bridge support brackets of Piers 1 through 6 Loss of support to extension pier legs, with collapse of the extension bridges likely.	• Crushing of diaphragm beam stiffeners and buckling of web plates in Piers 1 and 2.	❍	❍	Fit 200UC full depth bearing stiffeners to diaphragm beam, welded connection.
	• Buckling failure of diaphragm beam webs in Pier 3.	❍	❍	Bolt 200UC stiffeners to diaphragm beam
	• Failure of welds connecting diaphragm beam webs to bracket side plates in Piers 4, 5, and 6.	❍	❍	Reinforce existing welds to exceed strength of diaphragm beam plates using additional weld runs.
	• Failure of welds connecting bracket flange plates to side walls in Piers 1 and 2, 4, 5 & 6.	❍	❍	Strengthen connection using angle section welded to existing plates. Use "Wind Brace" to reduce response at Piers 1 & 2.
	• Buckling of bracket side walls in Piers 1 and 2.		❍	Use "Wind Brace" to reduce response and add intermediate plate stiffeners.
	• Failure of welds connecting bracket side plates to the back wall in Piers 4 & 6.		❍	Strengthen connection using angle section welded to existing plates.
	• Failure of anchor bolts connecting lower trestle bearings to brackets at Piers 1 and 2.		❍	Install "Wind Brace" to reduce response.
	• Buckling of bracket bottom flange in Piers 1, 2, 4 & 6.		❍	Use "Wind Brace" to reduce response at Piers 1 & 2. Add intermediate plate stiffeners at other piers.
	• Buckling of bracket back walls in Piers 1, 2, and 3.		❍	Concrete pilasters at Piers 1&2, additional plate stiffeners at Pier 3

Table 2 Major Vulnerabilities and Retrofit Solutions (Continued)

Description of Vulnerability, Potential Consequences	Nature of damage to critical components	Vulnerability		Retrofit Solution
		Seismic	LL/WL	
Extension bridge support trestles at Piers 1 through 6 Collapse of extension bridges at these piers, with global collapse likely.	• Failure of box column web-flange welds at Piers 4, 5 and 6, initiated by local plate buckling. • Buckling/flexural failure of box columns at Piers 1 and 2, initiated by failure of upper bearing keepers and/or column to bearing welds (inner edges). • Failure of upper bearing to box girder connection stiffener welds (inner edges) at Pier 3. • Failure of upper bearing to box girder welds and/ or stiffener welds (inner edges) at Piers 5 and 6.	❍ ❍	❍ ❍ ❍ ❍	Fit “anti-buckling” bars between compression flange and tension flange or webs. Install "Wind Brace" Marginal - inspect after major wind events, monitor wind loads and reassess. Retrofit difficult, inspect after major events, column retrofit design assumes cracked welds
Concrete pier walls - upper section adjacent to the steel extension brackets Concrete crushing, spalling, possible loss of support to steel brackets, possible extension bridge collapse.	• Shear failure at construction joints or shear/flexural failure at Piers 1, 2, 4, 5 and 6. • Bearing failure at bracket foot in Piers 1, 2, 4 and 5.		❍ ❍	Concrete pilasters to inside of upper pier walls. Piers 1&2 Wind Braces reduce demand Concrete pilasters, vertically post-tensioned overlay slab at Pier 4 only
Southern steel viaduct - end braking strut Collapse of northern end span of south approach. Loss of central four lanes.	• Failure of strut restraining deck panels, partly caused by longitudinal loading from adjacent watermain anchor point.	❍		Strengthen existing struts. Fit restrainers at Princes St Anchorage to avoid additional longitudinal load if bearing keepers fail.
Northern steel viaduct - earthquake strut Collapse of deck spans. Loss of central four lanes.	• Crushing of channel of built-up strut from water main pounding. Loss of longitudinal restraint to deck panels.	❍		Additional bracing to adequately restrain the watermain
Northern steel viaduct - Expansion joint @ Pier N6 Collapse of deck spans. Loss of central four lanes.	• Insufficient seat length for imposed ground displacements.	❍		Fit "Catch Frame" to support sliding end of deck strings if seating is lost.

Note: LL/WL Vulnerability denotes a Live Load or Wind Load vulnerability