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AUCKLAND HARBOUR BRIDGE SEISMIC ASSESSMENT

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ABSTRACT

The Auckland Harbour Bridge is a 1.6 km-long bridge spanning the Waitemata Harbour between the Auckland urban isthmus and the heavily populated North Shore in New Zealand. The original steel truss bridge was built in the 1950's but traffic growth required widening from four to eight lanes in the late 1960's. This was done using steel box girders supported on either side of the original piers. It is in an area of moderate seismicity, with the dominant earthquake sources in the range of 40 km to 100 km from the bridge. A preliminary seismic assessment has been carried out and a few but important vulnerabilities have been identified. The seismic assessment, vulnerabilities identified, and the action for the final assessment and development of retrofit alternatives are discussed in this paper.

KEYWORDS

Steel Bridge; Performance Criteria; Seismic Assessment; Seismic Retrofit; Seismicity; New Zealand

INTRODUCTION

The Auckland Harbour Bridge was constructed in the late 1950's to carry four lanes of traffic across the Waitemata Harbour between Auckland City and North Shore City. The bridge was widened in the late 1960's following several years of high traffic growth. The original bridge comprised the 1.0 km-long main steel truss bridge together with 0.6 km of approach viaducts of both steel and concrete construction. The main truss bridge has spans ranging from 80.1 m (265') to 244 m (800') and is supported on massive reinforced concrete cellular piers founded on sandstone bedrock.

The widening scheme (extensions) employed haunched steel box girders, one on each side of the original bridge, with steel trestle box piers supported on steel box extensions anchored to each side of the existing concrete piers. There are now effectively three separate bridges, on common piers, spanning the harbour. A general arrangement for the bridge is illustrated in Fig. 1.

Auckland is in a region of moderate seismicity, one of the lower seismicity areas in New Zealand. The original structure was design using a static uniform lateral seismic load of 0.10 g, in combination with a 1 kPa (20 psf) wind load and half of the design traffic load. A preliminary study of the extensions (WCS, 1993) identified several potentially serious seismic vulnerabilities, including pounding of the three bridges under transverse seismic loading and damage at the various articulations and joints in the main bridges and approaches.

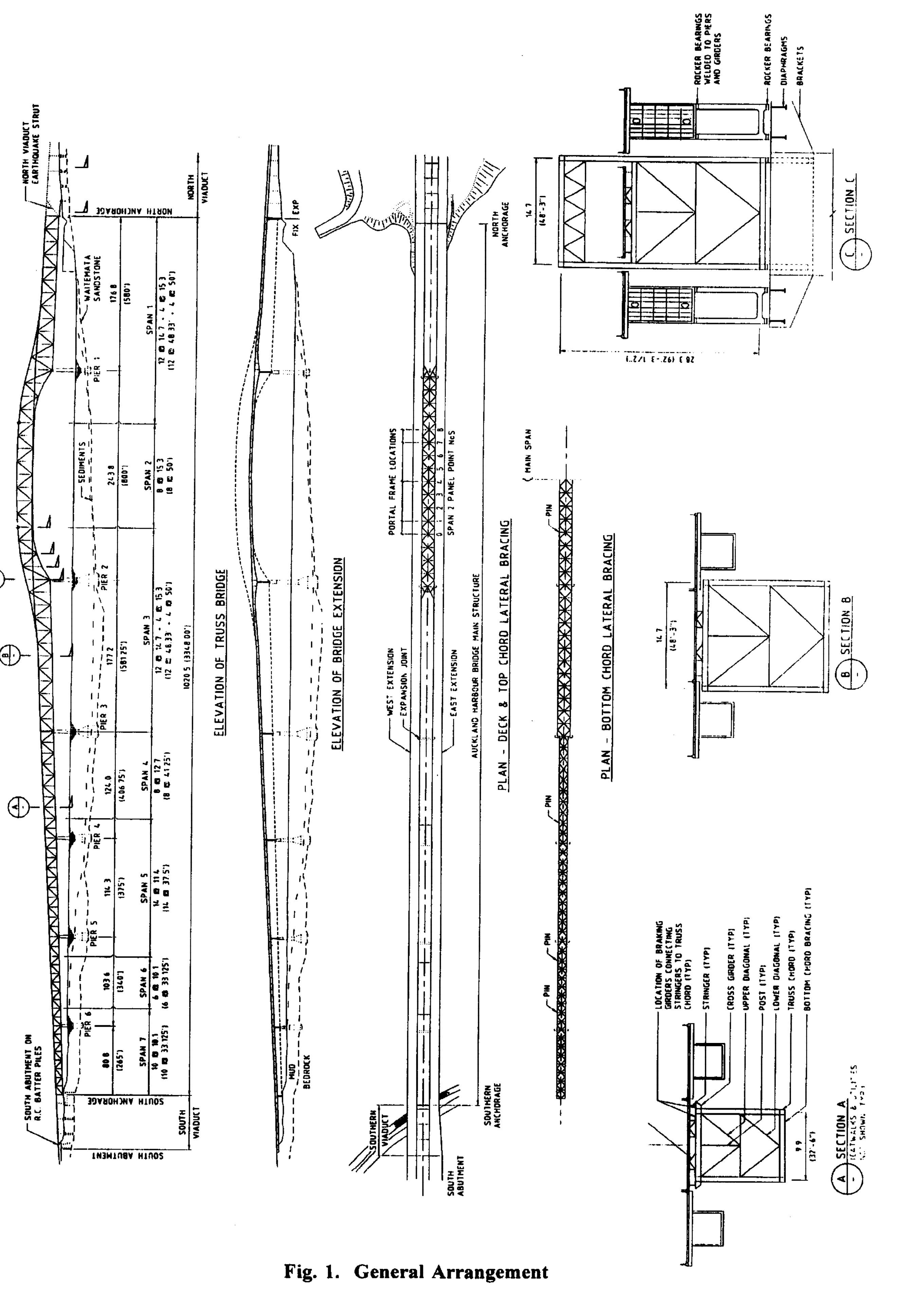

Fig. 1. General Arrangement

The bridge is an essential element in the State Highway 1 system linking the Auckland isthmus with the North Shore. Loss of use of the crossing due to earthquakes would have serious social and economic consequences for the Auckland region and beyond. Transit New Zealand commissioned the assessment and retrofit study, to be undertaken three stages. The first stage comprises a hazard study, a preliminary geotechnical assessment, and a structural assessment based on linear dynamic analyses. As well, order of magnitude retrofit costs and benefits are being developed. The second stage will comprise non-linear time history analyses as necessary, and focus on identified vulnerabilities. The third stage comprises design and economic analysis of seismic retrofit measures.

Specialist technical inputs were provided by Prof A. Astaneh-Asl (University of California, Berkeley), Prof M.J.N. Priestley (University of California, San Diego), Geomatrix (California), and the Institute of Geological and Nuclear Sciences (IGNS, New Zealand). The project is being peer reviewed by Works Consultancy Services (New Zealand). This paper discusses the first stage assessment. Subsequent stages are expected to continue during 1996.

PERFORMANCE CRITERIA

For a facility with the importance and complexity of the Auckland Harbour Bridge it is necessary to formulate the seismic performance standard considered appropriate at the onset of the project. This standard is defined by a set of performance objectives of the facility for corresponding levels of earthquake shaking. This becomes the standard against which the behaviour of the existing facility is assessed. To meet the performance standard, items which are assessed as vulnerable will require retrofitting.

The performance standard for the Auckland Harbour Bridge, comprising the objectives and the corresponding levels of earthquake shaking, are set out as follows:

Objective 1. In an earthquake shaking with a high likelihood of occurrence in the bridge life, damage and traffic disruption should be minimal.

Objective 2. In an earthquake shaking with an appropriately low risk of occurring in the life of the structure, the risk of loss of life should be at an acceptably low level and at least four traffic lanes should remain open. Damage should be readily repairable, allowing all eight lanes to be open in a few days.

Objective 3. In the maximum credible earthquake the structures should not collapse and major loss of life should be prevented. The bridge may be closed for some time and some permanent loss of function may result.

For the first stage of the study the following earthquake ground motions were adopted for each objective:

Objective 1 - 200 year return period. **Objective 2** - 2,000 year return period. **Objective 3** - Maximum credible earthquake (MCE) at mean plus 1.5 standard deviations. The MCE corresponds to a magnitude 7 earthquake with an epicentre 40 km from the site.

In developing these objectives and the corresponding earthquake ground motions, consideration was given to the relatively low site seismicity, to the standards adopted internationally for major bridges, and to the importance of the bridge to the post-earthquake recovery and to the Auckland economy. The performance standard chosen is relatively high for two main reasons. Firstly, the bridge is a key component in the economic and social welfare of the region. Secondly, the bridge was not expected to have many serious seismic deficiencies and the cost of retrofitting to this standard was expected to be relatively small. The ground motion levels for each objective will however be reviewed and possibly revised during the next stage, depending on the consequences of the vulnerabilities and the implied retrofit costs.

SEISMIC HAZARD ASSESSMENT

The site lies in an area of relatively low seismicity with the most significant fault identified as active being around 40 km away. The results of the seismic hazard assessment determine the ground motions and seismic loadings, and thus have a major impact on the assessment and retrofitting concepts.

IGNS carried out a regional seismicity study incorporating the latest available data. BCHF then developed seismic hazard spectra using probabilistic methods. Geomatrix reviewed the IGNS study, provided attenuation relationships for the hazard study, and reviewed the hazard assessment results.

BCHF developed uniform hazard spectra for different levels of earthquake shaking from 200-year up to 2000-year return periods, and also for the MCE. MCE response spectra were developed for varying confidence levels. Using the same hazard analysis program peak ground displacements were also derived. Maximum displacements of about 250 mm, roughly corresponding to the MCE risk level, were estimated and used as part of the structural assessment. The hazard spectra and MCE response spectrum are illustrated in Fig. 2.

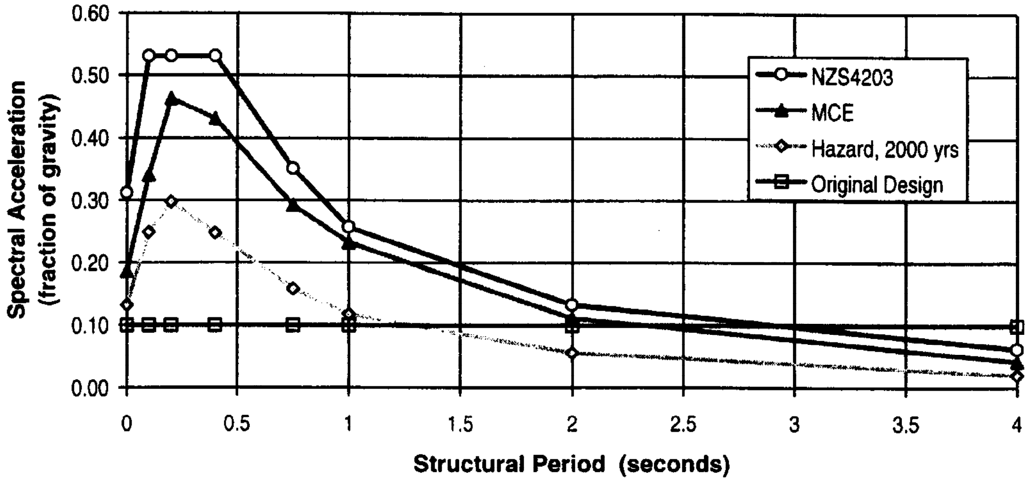

Fig. 2. Seismic Hazard and Response Spectra

The hazard spectra developed are much less that the seismic loadings given in NZS4203:1992. The BCHF 1000-year return period motion, for example, is about one third of the NZS loading. The MCE spectrum is slightly less than the NZS4203:1992 spectrum for a 1000 year return period motion.

ASSESSMENT METHODOLOGY

The seismic assessment and retrofitting of major bridges is a technology which has developed very rapidly in the last several years. The impetus for this development was the 1989 Loma Prieta Earthquake which resulted in massive damage to bridges in the San Francisco area. As a result, a major research effort and the development of appropriate methods to assess and retrofit such bridges was initiated in California. Professor Astaneh of UC Berkeley has been extensively involved in the testing and assessment of major steel bridges, and has been an important contributor to the on-going development of assessment methodologies. Similar comments apply to Dr Nigel Priestley of UC San Diego and concrete bridge assessment and retrofit. The assessment methodology used by BCHF for the Auckland Harbour Bridge draws heavily on the work of these two specialists, who are providing inputs and reviews to the project. The assessment in broad terms involves the following steps:

i) Assessing the actual material properties and condition of the various components.
ii) Analysing the structure to find the seismic forces and displacements.
iii) Assessing the components for their strength and ductility.
iv) Assessing the performance of critical components and the overall bridge performance against the chosen performance objectives.

Components which fail to meet the performance objectives set, are termed *vulnerabilities* and generally require retrofitting if the objectives are to be met.

The analysis and assessment involves obtaining data from an archive of thousands of drawings and reports, and requiring computer modelling of thousands of members. In addition it was believed that retrofitting, while critical, was likely to be confined to relatively few components. The interactions between the original truss bridge and the box girders is also expected to be highly non-linear, as were some connection and members. Accordingly, the study has been staged, with Stage 1 for the main bridges comprising elastic analyses. While this does not allow for the dynamic interaction ("pounding") between the truss bridge and extensions, it does however provide an appropriate tool for an initial assessment of the numerous bridge components, and giving an indication of the most likely vulnerabilities. Investigation of pounding and its consequences requires more sophisticated non-linear analyses, and these are proposed in Stage 2 to investigate efficiently the vulnerabilities indicated by the preliminary screening analyses.

The approach roads and approach viaducts did not require these complex analyses and so their final assessment has been carried out as part of stage 1, albeit with soil parameters based on a review of historical information.

Steel Member Assessment

The preliminary assessment of steel members was undertaken using provisions summarised in Astaneh (1995) the AISC (1990) and supplements, and the SSRC guide 1988. Built-up members were modelled and assessed using first principles, as described by the SSRC. A detailed review of the behaviour of built-up members by Caltrans (1995) appears useful for the truss bridge, and will be referenced during the final assessment.

Important aspects of the assessment of steel truss members for this project included fracture of members at net sections of connections, combined axial and flexural buckling of struts (angles, tees, and built-up members), and the available member ductility. The latter is significantly reduced by local member buckling modes for those members with high component plate slenderness ratios (Astaneh, 1995).

Important aspects of the steel trestle piers supporting box girder extensions included combined axial and flexural yield or buckling of unstiffened box columns and beams, force transfer across unstiffened beam-column joints of box members, and the available member ductility. The trestle piers were originally design using stiffened slender plates, but redesigned and constructed with stocky unstiffened plates, having b/t ratios in the range of 20 to 40 for the 250 MPa steel flanges, and 40 to 55 for the webs.

Concrete Member Assessment

The assessment of concrete in piers, foundations, and anchorages was undertaken using the established methodologies as outlined by Priestley et al (1992). Critical aspects of the concrete assessment included shear transfer across openings in the walls of the cellular main piers, shear in concrete piles, shear and tension in concrete walls of an approach span anchorage, and shear and flexural demands in concrete approach piers arising from relative ground displacement demands.

ANALYSES

Linear elastic dynamic analyses of three dimensional models were carried out for the truss bridge and the extensions. At this stage, separate models for the various bridge sections were used. Hand calculations and simplified dynamic analyses on foundation and substructure stiffnesses demonstrated that separate superstructure models would produce sufficiently accurate results for the initial assessment. SAP90 was used for the analyses, using the Ritz vector approach as opposed to the traditional mode shapes (eigenvectors). This proved particularly efficient for the truss bridge. Concurrent seismic effects from three directions were considered in the analyses. Damping for the welded box girder extensions was typically taken as 2% of critical, although for the assessment of transverse moments in the box girders, 5% of critical was assumed, based on initial results which indicated that pounding between structures, implying attenuation of these forces, would be expected. Damping for the main rivetted truss bridge dynamic analyses was taken as 5% of critical.

Steel Truss Bridge

Load Paths. An important aspect was to identify primary and redundant load paths for seismic loads, including those which may not have been considered in the original design. Lateral loads are resisted at each pier and at the north anchorage block. The primary transverse lateral load path as originally designed is via cross frames located at each panel point, to bottom chord bracing. The deck was not intended to transfer lateral loads. Each 15 m long (50') deck panel is fixed longitudinally via 'braking girders' to the truss chords at one end, and is free at the other. Panels are typically braced transversely by steel single angles at the fixed end. Despite intentions otherwise, this configuration forces the deck to participate as a diaphragm, inducing large axial forces in both braking girders and the single angle transverse braces, and significant demands in the connections of these members as well.

For longitudinal loads, the truss bridge is divided by an expansion joint at Pier 3, such that all of the longitudinal seismic forces are resisted at the south anchorage (from four approach spans) and the north anchorage (from three main spans). Concrete piers are isolated from longitudinal superstructure seismic loads by 6m tall rocker bearings.

Modelling. A detailed model was built using line elements at the centre-lines of all truss and bracing members. The truss models for the three span truss sections is shown in Fig. 3. The model accounted for the concrete deck, stringers and deck bracing members as described below.

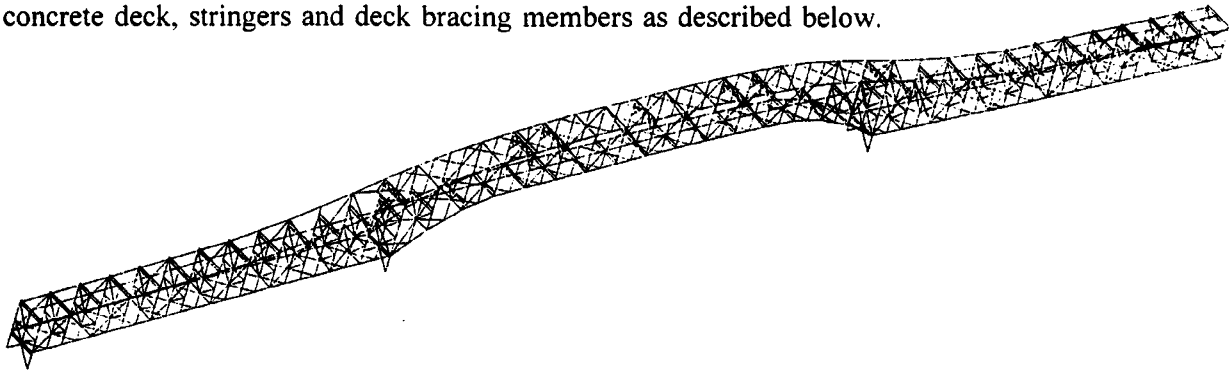

Fig. 3. Three Span Truss Model

Initially the deck panels were assumed to be free to slide at stringer-stringer breather joints and would have little stiffening effect on the response of the bridge. This is consistent with the designer's intent. Shearing displacements at the deck joints of approximately 35 mm were calculated. The connection details cannot accommodate deformations of this order without substantial damage. Hence was not realistic to ignore the stiffness of the concrete deck connections. To assess the effect of these constraints, a deck element was added to the model, supported by stiff links to the truss chord members. The effective stiffness of the deck elements at this stage was estimated using a sub-structure models and hand calculations. This aspect of the model will be included explicitly in the next stage. The fundamental transverse period of vibrations were found to be 3.5 seconds for the initial model, and 2.6 seconds for the more realistic 'stiffened deck' model. The first longitudinal and vertical modes had periods of 2.3 seconds and 1 second respectively in both models. There were no field measured data with which to compare these results.

To assess the sensitivity of the structure to out-of-phase ground displacements between piers, each pier in turn was displaced transversely by the maximum ground displacement obtained from the hazard study. While estimates of seismic wavelengths were made, as was the effect on relative pier displacements of various angles of incidence of the seismic waves, considerable uncertainty existed in these calculations. Therefore, the simple approach describe above was adopted to assess upper bound effects. These were generally significantly less than the forces from dynamic analyses.

Steel Box Extensions

Load Paths. Transverse loads are resisted at each pier, the south abutment, and the north anchorage. The box girder transfers transverse seismic loads to the supports in flexure in lateral bending, through upper rocker bearings to trestle piers, through rocker bearings to steel box extension brackets ('corbels') stressed onto the original concrete piers. Transverse seismic loads also induce vertical reactions to be resisted by the extension brackets, which exceed the governing demands induced by the original design wind loads.

All longitudinal loads are transferred as axial forces through the box girders to the north anchorage. At the north anchorage, pairs of steel box beam-columns framed integrally with the box girders to transfer the loads from deck level in bending down to the foundation. As a result, the box girders would also experience significant flexural demands from longitudinal seismic loads.

Modelling. The extensions box girders and piers were modelled using beam elements located at the neutral axis location along the length of the boxes. Torsional stiffness and demands were included in a simplified manner. Results indicated additional precision, by including warping torsional stiffness, was not required.

The north anchorage steel box legs were explicitly modelled. Other pier properties were developed from separate 3-dimensional frame models, and equivalent springs were incorporated into the global dynamic model. Out-of-phase ground movements were considered as described previously for the truss bridge.

The fundamental transverse and longitudinal modes had periods of 2.8 seconds and 3.7 seconds. Other important modes had periods below about one second. As expected, significant coupling was observed between longitudinal and vertical modes. The mode shapes and periods compared favourably with dynamic test results undertaken by Wood and Fowler (1976).

ASSESSED PERFORMANCE

The expected damage at each level of ground motion was assessed based on the analyses and described assessment methodology. Minor to moderate damage was assessed in numerous elements, but in most cases the performance standard was not violated by the predicted damage. A summary of the main damage at each level of ground motion follows.

At low levels of shaking with a return period of about 200 years, no significant damage is expected.

At moderately high levels of shaking with a return period of 1,000 to 2,000 years, the truss bridge is assessed as likely to be closed for a significant period due to collapse of deck panels within the truss bridge and span losses on the north approach viaducts. Damage to the southern abutment is predicted, and would require lane closures for some time during repairs.

For maximum credible earthquake shaking the truss bridge is expected to be closed due to loss of deck panels and span collapses on the north and south steel approach viaducts. Also, major damage to a number of truss bridge main members due to pounding (collisions) between the truss bridges and the extensions is also likely. Damage to the southern abutment may require it to be reconstructed. The truss bridge is expected to be closed for at least several months.

Assessed Vulnerabilities

General damage to the structure has been briefly outlined above. For a number of components, damage is predicted to be serious enough such that the performance objectives as described earlier in the paper cannot be met. These are classified as seismic *vulnerabilities*. Table 1 summarises these vulnerabilities.

Table 1. Seismic Vulnerabilities.

Item	Vulnerability	Consequences
Main truss bridge - Vertical post at Piers 1 & 2.	Compression/flexure buckling	Collapse of deck panels, major damage to trusses.
Truss bridges - Braking girders	Failure of girders restraining deck panels	Collapse of deck panels, damage to trusses.
Main truss bridge stringer bracing.	Net section fracture at connections	Transverse deformations of deck panels; local collapse.
Main Bridges	Pounding between bridges. Failure of truss diagonals	Major damage to trusses. Increased risk of assessed or potential vulnerabilities.
Southern steel viaduct - end braking girder.	Failure of girders restraining deck panels, in part from watermain loading.	Collapse of deck spans.
Northern steel viaduct - earthquake strut.	Crushing of channel of built-up strut from pounding with water main	Collapse of deck spans.
Northern steel viaduct - Expansion joint @ Pier N6.	Insufficient seat length	Span collapses

The sensitivity of identified vulnerabilities to reduced ground motion levels was also assessed. For a 1000-year return period hazard spectrum and an MCE at one standard deviation, the only component which would be eliminated as a vulnerability is the end braking girder connecting the southern viaduct to the main truss bridge for longitudinal seismic loads.

FURTHER PROJECT STAGES

The preliminary assessment using linear dynamic analyses has identified that the facility has a number of potentially serious vulnerabilities, including expected pounding between the truss bridge and the extensions. Subsequent stages will include non-linear time history analyses to include the effects of pounding between the three bridge structures, and to better define the extent of required seismic retrofits. Retrofit strategies and economic analyses of the alternatives will be undertaken prior to selection and detailed design of seismic upgrades.

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