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THORNDON OVERBRIDGE SEISMIC RETROFIT

IAN J BILLINGS AND ALAN J POWELL

Beca Carter Hollings and Ferner Ltd
PO Box 6345, Auckland, New Zealand

ABSTRACT

The Thorndon Overbridge is a 1.3 km long elevated concrete bridge located on the reclaimed foreshore of the Wellington Harbour in New Zealand. It is in an area of high seismicity with the dominant earthquake source, the Wellington Fault, passing under the bridge. The structure was designed prior to the 1970's and has serious seismic vulnerabilities. The proposed retrofit includes pilecap strengthening, column jacketing, portal pier infill walls, seat extensions, restrainer modifications and a fault 'catch frame'. The seismic assessment, proposed retrofit and economic analysis, are discussed in this paper.

KEYWORDS

Concrete Bridge; Seismic Retrofit; Seismic Assessment; Cost Benefit Analysis; Pilecap Retrofit

INTRODUCTION

The Thorndon Overbridge comprises twin 1.3 km three lane elevated concrete bridges located on the shore of the Wellington Harbour in New Zealand. It spans over the Cook Strait ferry terminal, an extensive area of rail yards and two other important roads, Hutt and Aotea Quays. On and off ramps mid way along access the Aotea Quay. The overbridge forms part of an important link from Wellington City to the north.

It was constructed in three stages between 1967 and 1972 with different substructure types in each stage. The superstructure consists of simply supported precast concrete I girders spanning between large pier cap umbrellas. Substantial linkage bolts tie the girders onto the umbrellas. The first substructure stage consisted of multi column framed bents on driven piles. The later stages utilised single column piers on either driven or bored piles. The bridge is located on reclaimed land placed between 1882 to 1970. The reclamations typically consist of 4m to 16m of gravel rockfill or pumped hydraulic fill and overlie a 1 to 2 m layer of sandy gravel Holocene beach and marine sediments. The bridge piles are founded below the beach layer, and have substantial steel casings in all cases. The bridge layout and typical single column piers are shown in Figs 1 and 2.

The Thorndon Overbridge is located in an area of high seismicity and in fact crosses over the Wellington fault, the dominant active fault in the area. A site specific seismic hazard study was carried out as part of the assessment and showed that for a one second period the design seismic acceleration is 1.5g for a 1000 year return period and 0.7g for a 500 year return period. The study also showed that there is a 67% chance that the next large magnitude earthquake at the site will occur on the Wellington Fault. Permanent ground displacement of approximately 5m horizontal and 1 m vertical can be expected from a Wellington Fault event. The original structure was designed for a seismic acceleration of .3g and typically the seismic detailing improved with each stage of construction. The structure was not designed using the "capacity design" concept of modern codes and consequently has serious seismic vulnerabilities.

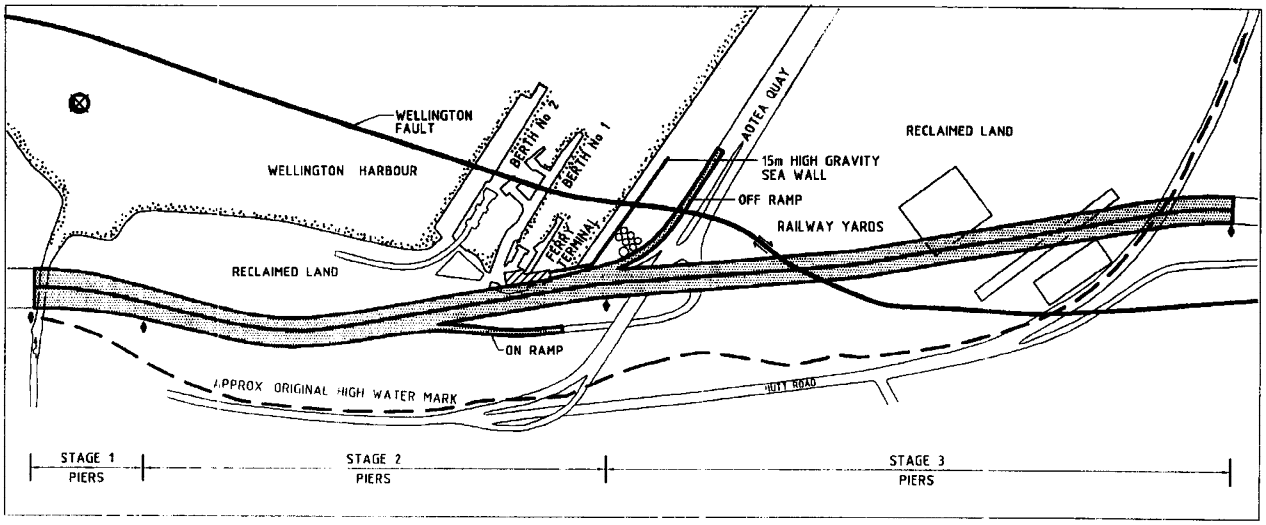


Fig. 1 Thorndon Overbridge Plan

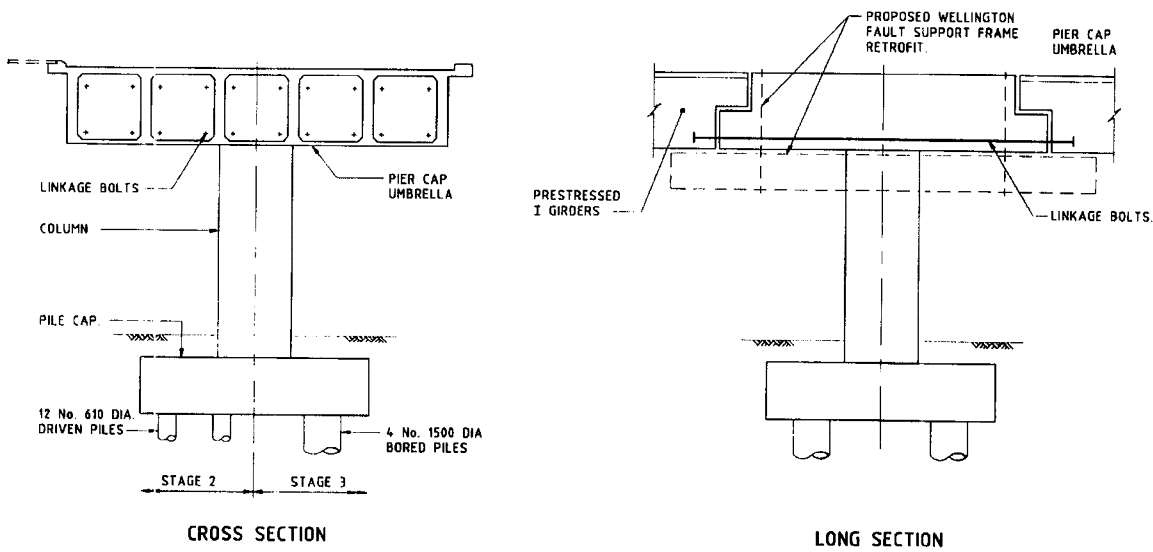


Fig. 2 Typical Existing Single Column Piers

The retrofit project, commissioned by Transit New Zealand, consists of assessment, retrofit concept development, and detailed design phases. A special requirement of Transit New Zealand was that detailed economic analyses of several proposed retrofit standards be carried out as this is required for all their projects. Detailed cost/benefit studies were carried out using probabilistic analysis techniques. An added bonus was the resultant data on likely fatality levels. Specialist technical assistance was provided by the Institute of Geophysical and Nuclear Sciences, Prof. Dr G Martin, University of Southern California and Prof. Dr M J Nigel Priestley, University of California, San Deigo. The project was Peer Reviewed by Works Consultancy Services.

This paper presents a summary of the assessment and retrofit concepts phases. Detailed design is being completed in stages, the first of which is now complete.

ASSESSMENT PHASE

A detailed discussion of the seismic assessment of the Thorndon Overbridge is included in Ref [2]. The following provides a summary.

Performance Criteria

Transit New Zealand's earthquake performance objectives for the Thorndon Overbridge were set out in qualitative terms and included the following:

1. At low levels of ground shaking the bridge should remain servicable.
2. The level of ground shaking which results in permanent ground displacements (due to liquefaction) should be identified so that the site and resultant bridge performance are understood.
3. At high levels of ground shaking the risk to life should be minimised.

As predefined earthquake levels against which to assess the bridge performance were not given, the strength and ductility capacity and therefore expected performance of each element of the bridge was assessed in terms of the ground shaking corresponding to a particular earthquake return period. Using these data together with a study of appropriate risk levels, return periods of ground shaking were assigned as:

Objective 1	-	50 year return period of ground shaking
Objective 2	-	200 year return period of ground shaking
Objective 3	-	300 to 1000 year return period of ground shaking

The performance was assessed specifically at 300 and 500 year return periods because these correspond to thresholds of major damage (and consequent retrofitting costs).

Assessment Methodology

Research on assessment procedures for existing reinforced concrete bridges within New Zealand has been limited thus the assessment procedure for the Thorndon Overbridge was largely based on Californian research. [1] The procedure seeks to determine the strength and ductility of the critical collapse mechanisms for the structure. The assessment methodology is summarised below.

- Assess probable material and section properties
- Analyse the structure to assess seismic spectral response demands.
- Assess member strength and overstrength capacities.
- Assess ductility capacities for members undergoing inelastic action.
- For each member's strength or displacement capacity calculate probable earthquake motion return periods by comparing the capacities to either the spectral response demands using the equal displacement theorem (structural period is typically 1 second) or to the assessed peak ground displacements.
- Check that all load paths are capable of transferring the inertia forces from the assessed substructure member strengths.
- Assess the performance of the bridge with respect to the following events:
  - Loss of soil strength due to liquefaction
  - Lateral movements of the foundations due to liquefaction and seawall failure
  - Gross deformations of the Wellington fault under the bridge
  - Differential longitudinal movements at expansion joints resulting from vibrational response characteristics and non-synchronous ground motion

Analysis Procedures

The overbridge contains two superstructure expansion joints per span which are able to rotate in plan and consequently the piers tend to respond independently to seismic forces in the transverse direction. Typical individual piers were analysed using soil structure models to account for the flexibility of the foundation materials. Dynamic modal analysis of sections of the bridge were performed to prove the independence of the various pier types. For the stage 1 multi column piers a push over analysis was used.

In the longitudinal direction the structure response is more difficult to model. The problem was bounded by assuming the piers were either independent or tied together throughout the bridge length. Both response spectrum and travelling seismic wave ground displacement analyses were carried out.

Typically the out of phase ground displacements resulted in the most critical demands on the bridge components. However these were not significantly different to demands obtained from the response spectrum analysis.

Pilecap Strength and Ductility

Typically the pilecaps are the limiting mechanism under seismic loading and therefore estimating their strength and ductility capacity was important for the assessment of overall bridge performance. The columns typically contain 2% to 6% longitudinal reinforcement and welded stirrups. The pilecap strength capacity was calculated using "strut and tie" modelling techniques with a number of potential failure surfaces being investigated.

Traditional column or beam ductility calculations were not considered applicable because the flexural reinforcement is unconfined. In addition, at a number of the pile caps tension only yielding of the bottom reinforcement occurs with a subsequent ratchetting downward movement of the column. Based on recommendations from Dr Nigel Priestley a procedure for estimating pilecap ductility was developed.

In calculating the ductility capacity the limiting reinforcement tensile strain was taken as 0.05 to reduce the probability of bar buckling and crack widths under reversal of loads. Since the length from the contraflexure point to the maximum moment location is small in the pilecaps the plastic hinge length is dominated by strain penetration. This was taken as .022 f_y d_b (6 d_b) [1]. As strain penetration will occur in both directions from the flexural crack, compared to one direction from a column/beam interface, a plastic hinge length of twice the strain penetration (12 d_b) was used. From these criteria a plastic rotation in the pile cap was calculated, the plastic displacement at the pier cap level calculated without further allowance for flexural curvature, and the ductility then calculated. To account for the tension only yielding of some of the pile cap reinforcement, reduction factors were applied.

Assessed Performance

The earthquake return period capacities for all bridge and site elements were summarised and then the overall bridge performance assessed against four earthquake return periods. These earthquake return periods were chosen to best satisfy Transit New Zealand's earthquake performance objectives for the Thorndon Overbridge as discussed above.

The performance of the Thorndon Overbridge is summarised for the four earthquake return periods as follows.

50 Year Return Period Earthquake Event. Damage to the existing bridge structure is relatively minor and it is assessed as remaining fully serviceable.

200 Year Return Period Earthquake Event. Liquefaction of the sands located under and seaward of the off-ramp, leading to gross seaward movement in this area and collapse of the off-ramp. Elsewhere sufficient liquefaction of the old beach layer is expected to have occurred, resulting in permanent displacements at ground level of up to 25 mm. These would result from sliding block type failures with blocks adjacent to the sea, moving toward the sea generally transverse to the overbridge and off-ramp.

Damage is assessed as likely to a significant number of pile caps with major damage to 15 Stage 1 and Stage 2 pier pile caps necessitating urgent repair work and temporary propping. The bridge is assessed as remaining serviceable providing securing and repairs are instigated immediately.

300 Year Return Period Earthquake Event. Widespread liquefaction leading to major seaward movement of the reclaimed lands of about 150mm. Serious damage to all *Stage 1 and 2* pier pilecaps with some loss of gravity support. Collapse of the offramp and loss of seating at the on ramp is expected. There is a *moderate* probability of occurrence of movement on the Wellington Fault with subsequent span collapses.

500 Year Return Period Earthquake Event. Seaward movement of the reclaimed lands of about 500 m. Serious damage to all *Stage 1, 2 and 3* pier pilecaps with some loss of gravity support, collapse of the offramp and loss of seating at the on ramp is expected. There is a *high* probability of occurrence of movement on the Wellington Fault with subsequent span collapse. Note the seaward movement of the reclaimed lands is assessed to be approximately 1000 mm in a 1000 year return period event and to require ground retrofitting to prevent pile failure.

RETROFIT CONCEPTS PHASE

Retrofit Schemes

To address the vulnerable areas identified during the seismic assessment, three basic retrofit schemes designed for 500 year, 300 year, and 200 year return period earthquake forces were developed. These schemes are designated I, II and III respectively. To provide a basis for deciding on an appropriate level of retrofit a comparison of the economic benefits and performance of the various schemes was carried out. To assist with the assessment of appropriate risk a detailed risk study drawing on local and international data was performed. The retrofitting for the three basic schemes is summarised below.

Area of Structure		**Retrofit Scheme I**	**Retrofit Scheme II**	**Retrofit Scheme III**
Superstructure Linkages		Retrofit linkage bolts at all piers, seat extensions at ramps and abutments.	Retrofit linkage bolts at 20 piers, seat extensions at ramps.	No retrofit
Wellington Fault		Support frames at main structure & off-ramp.	No retrofit	No retrofit
Foundations and Columns at Main Structure	**Stage 1 Piers** (9 piers total)	Steel column jackets. Infill concrete walls. Pilecap overlays.	Steel column jackets and pilecap overlays.	Steel column jackets and pilecap overlays.
	Stage 2 Piers (35 columns and 31 pilecaps, total)	Steel jacket on all columns. Overlays on all pilecaps. Post-tension 27 pilecaps. New piles at Pier 19.	Steel jackets on 23 columns. Overlay on 28 pilecaps. Post-tension 10 pilecaps.	Steel jacket 3 columns. Overlay 10 pilecaps. Post-tension 7 pilecaps.
	Stage 3 Piers (Area south of Aotea Quay - 32 columns and pilecaps, total)	Steel jackets on 2 columns. Overlays on 14 pilecaps. Post-tension 18 pilecaps.	Steel jackets on 2 columns. Overlay on 4 pilecaps. No post-tensioning.	Overlay on 4 pilecaps.
Off-ramp	**Piers**	Overlay on 7 pilecaps.	No retrofit	No retrofit
	Ground Improvement	Stone columns in soil	Stone columns in soil	Stone columns in soil
On-ramp		Overlay on 4 pilecaps	No retrofit	No retrofit
Estimated Cost (Ex GST)		NZ$19 m	NZ$9 m	NZ$7 m

In addition to the three basic retrofit schemes described above, a further scheme, which has a ground shaking design level corresponding to a 1000 year return period and includes for retrofitting of the major seaward movements of the site, was developed. This scheme, designated Scheme I - plus Ground Improvement, essentially comprises all the components of Scheme I above plus a major ground improvement to mitigate against seaward movement of the reclaimed lands under the bridge. The estimated cost of Scheme I-plus Ground Improvement was NZ $59 million.

As discussed in the Assessment Phase the key areas of vulnerability are the pier pilecaps and span collapses over the Wellington Fault. The retrofit measures for each of these key items are discussed below.

Piers. To provide a reliable seismic performance, bridges are typically designed to allow "plastic hinging" of the columns. This requires the column strength to be less than the pilecap, the opposite to the existing condition of the Thorndon Overbridge. Therefore typically the retrofit schemes, with the exception of the lowest level of retrofit, improve the pilecap strength, and thereby force plastic hinging of the columns.

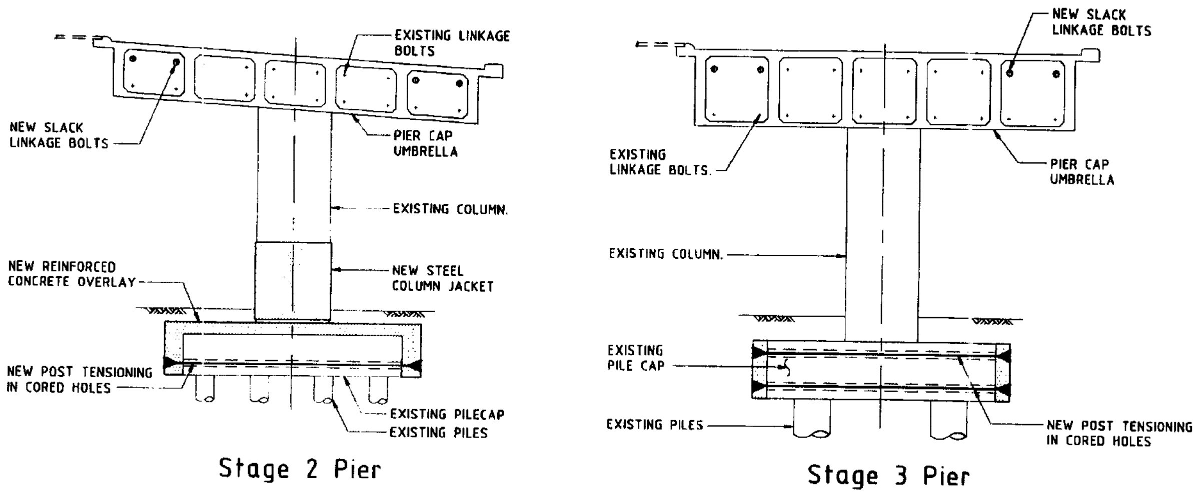


Fig. 3 Single Column Pier Retrofits

The pilecap strengthening is achieved by using a reinforced concrete overlay, or a cored through post-tensioning, or a combination of both. These retrofits are illustrated in Figure 3. The reinforced concrete overlays are connected to the existing pilecaps using drilled and grouted dowels which are designed assuming a shear-friction mechanism. Post-tensioning is added to the existing pilecaps by excavating at each end of the pilecap and coring holes through the length of the pilecap. The post-tensioning strands are placed through the cored holes and anchored into new reinforced concrete end blocks. For the Stage 3 piers overlay thickness is restricted by the rail tracks that pass over them.

The Stage 3 pier columns which contain 20 mm stirrups at 100 mm centres have sufficient confinement for plastic hinging but the Stage 1 and 2 pier columns which contain 12 mm stirrups at 300 mm centre require steel jacketting in the plastic hinge zones.

Wellington Fault. The Wellington Fault has the potential to cause a 5 metre ground offset where the main bridge structure passes over it. A retrofit concept was developed to prevent collapse of the superstructure, should the movement occur on the Wellington Fault. The retrofit consists of eight frames, built up of steel beams, secured to the pier umbrellas by vertical post tensioning. Several of the existing linkage bolts are replaced with slack restrainers which allow equal movements to occur at each expansion joint. The frames are designed to support the superstructure once it is pulled off the pier umbrella seats. The steel frames are located in spans crossing the fault and immediately adjacent, to allow for uncertainties in the assessed fault location. Fig. 2 shows the proposed support frames.

Laboratory Testing

Laboratory testing of two of the pier retrofit concepts is currently being undertaken by the University of Canterbury. The first test will be of a pilecap overlay only retrofit for a Stage 3 pier. These piers rely on pilecap strength and ductility to meet the design force or displacement levels. The testing purpose is to verify the pilecap ductility assessment methodology explained earlier in this paper. The second test will be of the typical Stage 2 pier retrofit shown in Figure 3. These piers rely on column strength and ductility to meet design force or displacement levels. The testing purpose is to verify the pier retrofit proposed, and provide an indication of the column overstrength to ideal strength ratio. This will assist in the design of the pilecap retrofit.

Cost Benefit Analysis

Transit New Zealand policy requires an economic evaluation to be undertaken for capital works projects. This is done using a cost benefit analysis (CBA). A unique feature of the Thorndon Overbridge CBA is that all benefits are probabilistic. The probabilities of occurrence which were taken into account included items such as: earthquake occurrence and assessed damage for all relevant earthquake sources; probability of peak, interpeak or night time traffic flows coinciding with an earthquake; probability of ferry arrival coinciding with an earthquake for either peak, shoulder or low season; probability of various span collapse scenarios.

The analysis was carried out using a spreadsheet and risk analysis software which allowed, a Monte Carlo simulation to be carried out for a large number of calculations, taking account of the above probabilities of occurrence. Outputs from the CBA were instrumental in the decision making process as they clearly showed the expected performance and benefits of the various retrofits.

Expected Performance and Benefits of Retrofit Schemes

In choosing an appropriate retrofit scheme for the Thorndon Overbridge the benefits of each scheme were carefully studied. The schemes were evaluated with respect to (a) functionality and safety levels, (b) potential fatalities and economic losses, and (c) benefit cost ratios. Typical performance data are indicated in the Figure 4 graph.

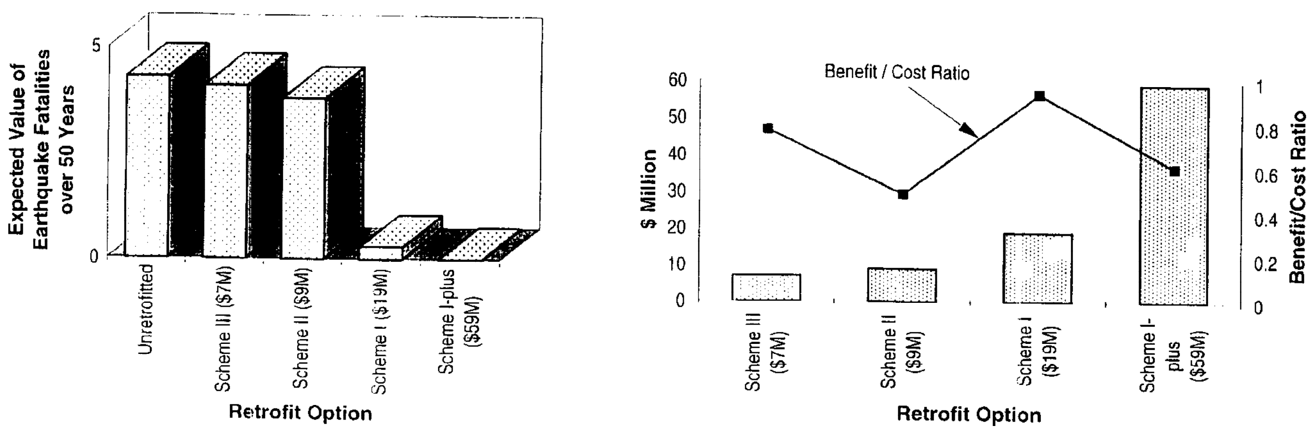


Fig. 4 Expected Fatalities and Benefit Cost Ratios

The off-ramp is expected to fail at relatively low levels of ground shaking, but use of the main bridge can continue without the off-ramp. Retrofit of the off-ramp is expensive and in the event of a Wellington Fault rupture it will still require replacement, even if retrofitted, because of the level of damage it will suffer. For these reasons the benefits of off ramp retrofitting are small.

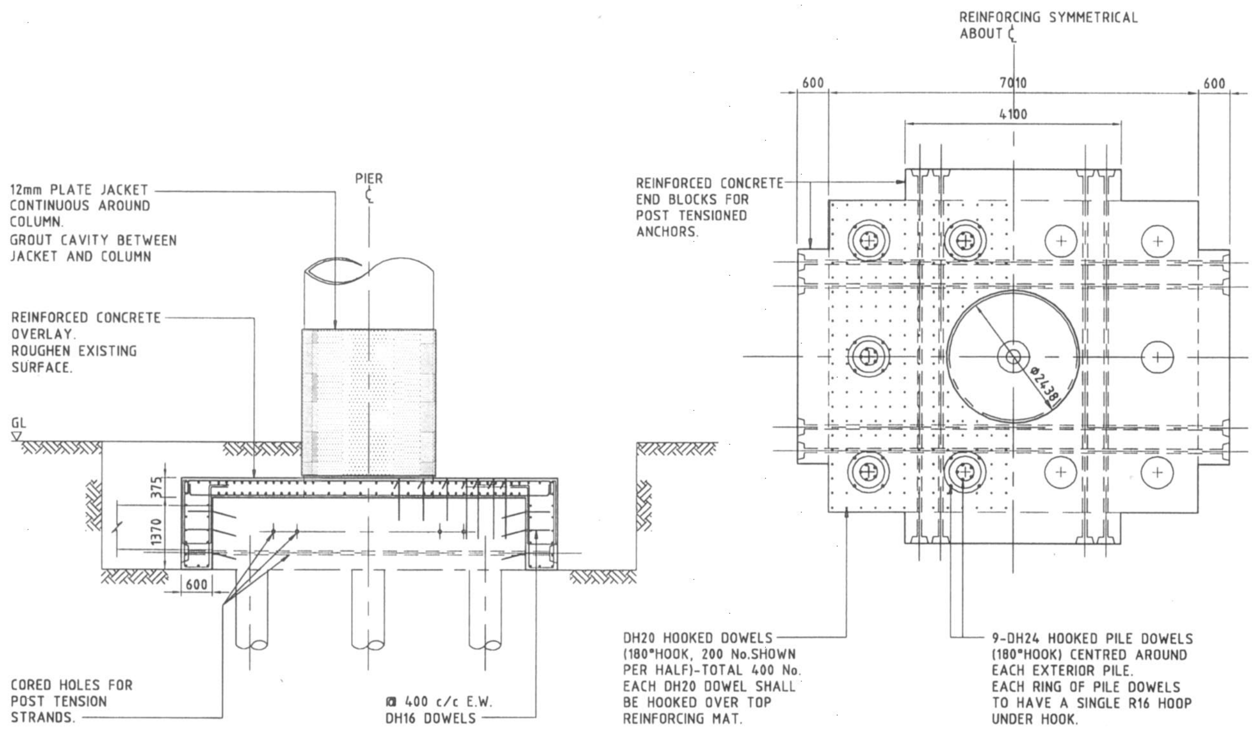
The expected values of fatalities are 4 for an unretrofitted bridge and 1 for a Scheme I retrofit. These appear low but this is somewhat misleading because they account for the probability of the earthquake actually occurring. In deriving these values worst case scenarios for earthquake fatalities were estimated. An example is, if a Wellington fault earthquake occurs during peak traffic hours and during the peak arrival time for ferry passengers, 100 to 150 persons could be killed and 300 to 500 could be seriously injured due to bridge collapse. Obviously time of day has a large influence on the expected number of casualties. The benefit cost ratios provide a relative measure to compare retrofit options. Scheme I and Scheme I plus Ground Improvement significantly improve the performance of the facility and also provide a significant reduction in deaths and economic losses. Scheme I plus Ground Improvement has a larger effect on reducing economic losses but its retrofit cost is much higher than Scheme I and consequently, it has a lower benefit cost ratio. Scheme I has a benefit cost ratio of about 1, the highest benefit cost ratio of all retrofit schemes. By leaving out the off ramp retrofit, the benefit cost ratio for Scheme I improved to about 1.3.

Chosen Retrofit Scheme

In reaching a conclusion for an appropriate retrofit level for the Thorndon Overbridge the above factors plus the results of a separate seismic risk study were taken into consideration. This study, investigated accepted risks and design levels for bridges in other jurisdictions (namely North America), new structures, new and existing facilities in Wellington, and by society generally for a variety of hazards. Scheme I, excluding the offramp retrofit, has been approved for detailed design and construction by Transit New Zealand.

Design Details

The detailed design of the first stage of the project (for which the laboratory testing results are not required) has been completed. A typical single column pier retrofit is illustrated in Figure 5.



SECTION - PIER GROUP F

PLAN - PIER GROUP F

Fig. 5 Typical Single Column Pier Retrofit

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