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THORNDON OVERBRIDGE SEISMIC RETROFIT

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SUMMARY

The Thorndon Overbridge is a 1.3 km long concrete bridge located on the reclaimed fore shore 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. This paper describes the seismic assessment and development of retrofit concepts which have been undertaken for this bridge.

INTRODUCTION

The Thorndon Overbridge is a 1.3 km twin three lane bridge located on the shore of the Wellington Harbour in New Zealand. It spans over the Cook Strait ferry terminal and an extensive area of rail yards. 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. 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 structure is very similar in form to the section of the Hanshin Expressway which collapsed in the recent earthquake near Kobe, Japan. The bridge layout and typical single column piers are shown in Figures 1, 2 and 3.

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. 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.

The retrofit project, commissioned by Transit New Zealand, consists of assessment, retrofit concept development, and detailed design phases. The first two are complete and detailed design is underway.

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.

ASSESSMENT PHASE

SEISMICITY

A site specific seismic hazard study was carried out for this project. This study showed that for an earthquake return period of 475 years the seismic design forces at this site, are similar to those obtained from the New Zealand Seismic Loadings Standard (NZS 4203:1992) and significantly higher for return periods larger than 475 years.

The Wellington Fault, which passes under the bridge, has a dominant effect on the site seismicity. The study shows that there is a 67% chance that the next large magnitude earthquake at the site will occur on the Wellington Fault. Permanent ground displacements of approximately 5m horizontal and 1 m vertical can be expected from a Wellington Fault event.

In addition to providing hazard spectra the seismic study estimated peak ground displacements caused by travelling seismic waves. These varied from approximately 150 mm for a 100 year return period to 500 mm for a 1000 year return period.

GEOTECHNICAL CONDITIONS

The ground reclamations in the vicinity of the overbridge were carried out in stages, primarily in 1882, 1904, 1924-32, 1960's and 1970. These are shown on Figures 1 and 2. The reclamations typically consist of 4 m to 16 m 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 in Pleistocene sediments below the beach layer.

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A 15 m high mass concrete seawall supports the 1924-32 reclamation, but much of this was covered by subsequent reclamations.

Investigations using machine boreholes and cone penetrometers were undertaken primarily to assess the liquefaction potential of the site. The investigation established that the beach which extends throughout much of the site is potentially liquefiable. In addition an area of liquefiable clean sands was located at the offramp. Elsewhere only minor lenses of liquefiable material were discovered. The performance of the site under seismic shaking is discussed later in this paper.

BRIDGE STRUCTURE

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. This 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 between 0.85 to 1.15 seconds) 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. Consequently the piers tend to respond independently to seismic forces in the transverse direction. The piers were analysed using soil structure models to account for the flexibility of the foundation materials. The effects of the large pier cap umbrellas were investigated using dynamic response spectrum analyses. For the portal 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 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.

Bridge Member Strengths

An important aspect of any seismic assessment is to obtain realistic estimates of member behaviour. Code guidelines are not appropriate for this because they are based on dependable behaviour rather than probable behaviour. The probable strength and ductility capacities for the Thorndon Overbridge elements were mainly calculated using models proposed by Priestley et al [1] [2] [3] [4] [5].

One exception was the ductility of the pilecaps. Experimental investigation of the behaviour of non retrofitted pilecaps is limited and consequently first principles were used to assess pile cap capacities for the Thorndon Overbridge. Typically the pile caps are the limiting mechanism under seismic loading and therefore estimating their strength and ductility capacity was important for the assessment of the overall bridge performance. The pilecaps are typically heavily reinforced on the bottom and lightly reinforced on the top. Generally the pilecap and column reinforcement is well anchored. The pilecap strength capacity was calculated using "strut and tie" modelling techniques with a number of potential failure surfaces being were 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 maximum 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.

The assessed ductility of yielding pilecaps for the Thorndon Overbridge is currently being verified by laboratory testing at the University of Canterbury.

ASSESSED PERFORMANCE

Seismic Event Levels

Transit New Zealand's specified earthquake performance objectives for the Thorndon Overbridge were set out in qualitative terms only. Based on these objectives three target performance levels, designated as Event Levels, were set. The performance of the bridge and site was measured against the Event Levels using the calculated structure integrity earthquake return periods. These are defined as the point at which calculated spectral response demands or ground displacements exceed assessed capacities. The Event Levels chosen were:

Event Level 1 - "Serviceability" level earthquake. 50 year return period (RP) event.

Event Level 2 - Level at which permanent ground displacements commence. 200 year RP event.

Event Level 3 - Large seismic event in which risk to life should be minimised. This event level is considered to be the level to which it would be desirable to retrofit to if economically and technically feasible. For the assessment study two return period levels were chosen to bound the likely range considered appropriate for Event Level 3.

Event Level 3A - Defined as items expected to have a structure integrity limit exceeding 300 year RP but maintain gravity support in a 500 year RP event.

Event Level 3B - Defined as items expected to have a structure integrity limit exceeding 500 year RP but maintain gravity support in a 1000 year RP event.

Assessed Performance

The performance of the Thorndon Overbridge is summarised for the three event levels (EL) as follows. Pier numbers are as shown on Figures 1 and 2.

Event Level 1 (50 year RP)

Damage to the existing bridge structure is relatively minor and it is assessed as remaining fully serviceable.

Event Level 2 (200 year RP)

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 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 I and Stage II pier pile caps necessitating urgent repair work and temporary propping. The bridge is assessed as remaining serviceable providing securing and repairs are instigated immediately.

Event Level 3

Widespread liquefaction leading to major seaward ground displacements expected to occur with movements ranging from about 150mm for a 300 year RP event to more than 1000mm for a 1000 year RP event. The movements are expected to be slightly larger toward the northern end of the site.

Major damage to significant portions of the bridge together with pier collapses and span losses are assessed to occur for both EL3A and 3B. As would be expected the performance under EL3A is assessed as being considerably better than EL3B. In an EL3A or 3B there is a high risk of loss of life and the bridge will become unserviceable for a long period of time while major sections of the structure are rebuilt.

Critical performance items are assessed as follows:

EL 3A (300 year RP)

- Off-ramp piers 1A-9A - possible collapse
- On-ramp/overbridge joint - loss of seating
- Piers - serious damage to all stage I and II piers with loss of gravity support to some.

- Wellington Fault movement under bridge - moderate probability of occurrence with subsequent span collapses.

EL 3B (500 year RP)

- Off-ramp piers 1A-9A - possible collapse
- On-ramp/overbridge joint - loss of seating
- Piers - serious damage to all piers except 1B to 4B (on/ramp) with some loss of gravity support to some.
- Wellington Fault movement under bridge - high probability with subsequent span collapse.
- Seawall failure - Pier 19 collapse

RETROFIT CONCEPTS PHASE

RETROFIT SCHEMES

To address the vulnerable areas identified during the seismic assessment, three basic schemes providing varying levels of retrofitting were developed. 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 following basic retrofit schemes were developed. These generally corresponded to the Event Levels 2, 3A and 3B discussed above.

Retrofit Scheme I: An *extensive* retrofit scheme designed for a ground shaking level corresponding to a 500 year return period, and designed to mitigate against collapse of the bridge due to movement of the Wellington fault.

Retrofit Scheme II: A *partial* retrofit scheme designed for a ground shaking level corresponding to a 300 year return period.

Retrofit Scheme III: A *minimal* retrofit scheme designed for a ground shaking level corresponding to a 200 year return period.

The retrofit measures and estimated cost developed for the three basic schemes are 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 I 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 II 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 III 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)		$19 m	$9 m	$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 block sliding of the site under the bridge. The estimated cost of Scheme I-plus Ground Improvement was $59 million.

As discussed in the Assessment Phase the key areas of vulnerability are the pile caps, span collapses over the Wellington Fault and collapse of the off-ramp because of liquefiable ground conditions. The retrofit measures for each of these key items are discussed below.

Pilecaps

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.

The pilecap strengthening is achieved by using a reinforced concrete overlay, cored through post-tensioning, or a combination of both. These retrofits are illustrated in Figure 4. 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.

An important consideration in developing the pilecap retrofits was the extensive areas of existing rail tracks. These either run over the pilecaps or adjacent to them. This restricted the depth of overlay which led to the extensive use of post tensioning, a less economic solution. In addition the close proximity of the tracks requires the use of sheet piling for excavation.

An investigation of columns undergoing plastic hinging showed that the Stage I and II columns have insufficient confining reinforcement in the plastic hinge zones. They contain 12 mm stirrups at 300 mm centres compared to the Stage III piers which contain 20 mm stirrups at 100 mm centres. The higher level retrofits utilize steel column jackets in the hinge zones for the Stage I and II piers. The use of fibre glass wrap as an alternative to the steel jackets will be investigated in the detailed design phase.

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.

Off ramp

Retrofit to prevent collapse of the off ramp requires the improvement of the potentially liquefiable sands on which it is founded. Various ground improvement schemes were investigated. These included stone columns, displacement piling, compaction grouting, contiguous concrete piles, jet grouting and diaphragm walls. Stone columns were selected as they are the cheapest solution. Stone columns are installed by vibrating and driving a closed tube to firm material and then compacting gravel in its place as the tube is withdrawn, thus forming a 'Stone column'.

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 III 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 II pier retrofit shown in Figure 4. These piers reply 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 are.

- earthquake occurrence and assessed damage for all relevant earthquake sources.
- probability of peak, interpeak or night time traffic flows coinciding with an earthquake (for road and mainline rail traffic).

- probability of ferry arrival coinciding with an earthquake for either peak, shoulder or low season.

- probability of span collapses in various areas - several scenarios are possible, each having different outcomes.

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 5 graphs.

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 may 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. It can be seen that Scheme I and Scheme I plus Ground Improvement significantly improve performance and reduce deaths. They also provide a significant reduction in 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 was recommended as the preferred retrofit level for the Thorndon Overbridge.

Scheme I, excluding the offramp retrofit, has been approved for detailed design and construction by Transit New Zealand. (The New Zealand Government Land Transportation Authority).

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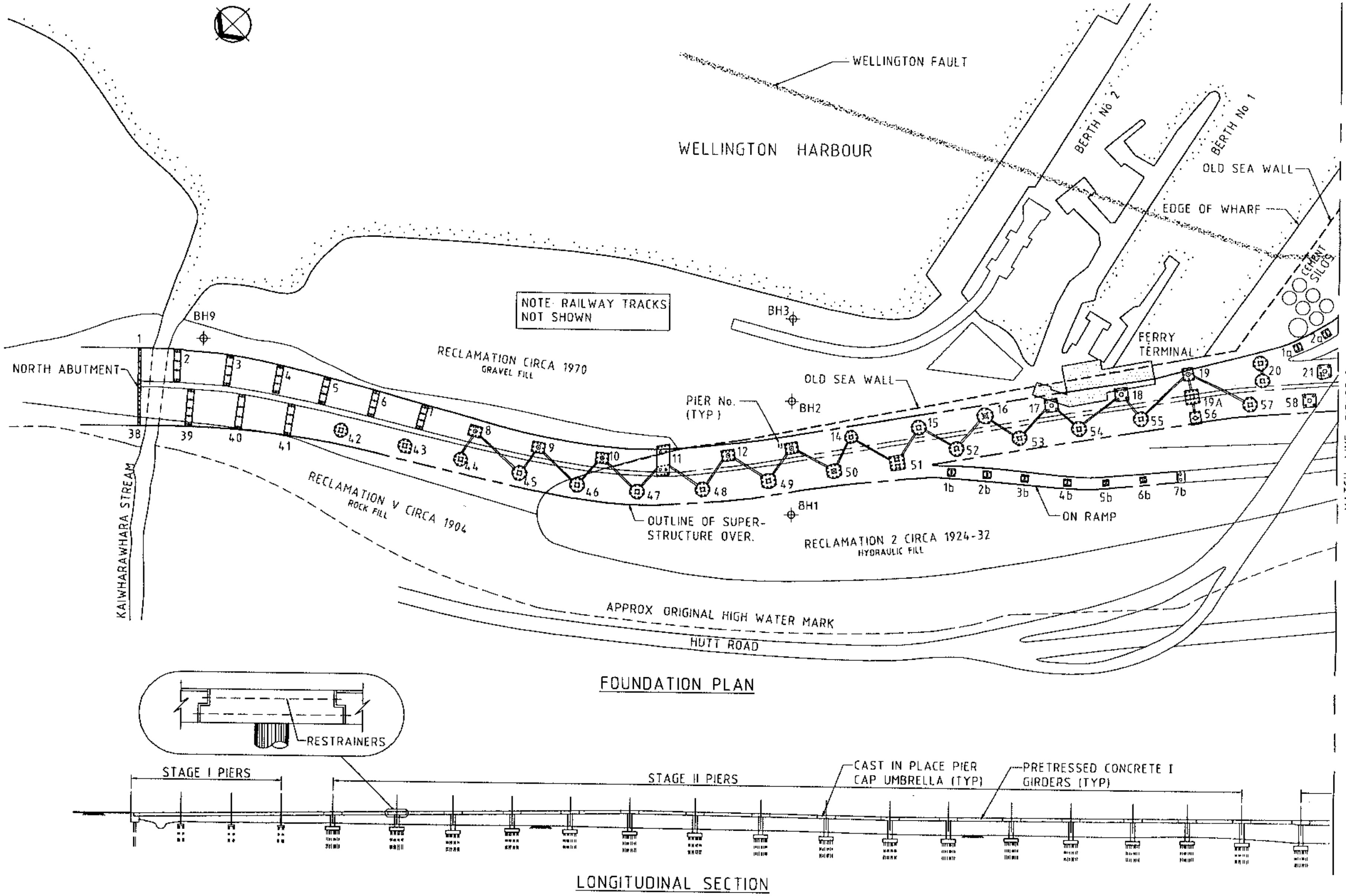


FIGURE 1 - THORNDON OVERBRIDGE

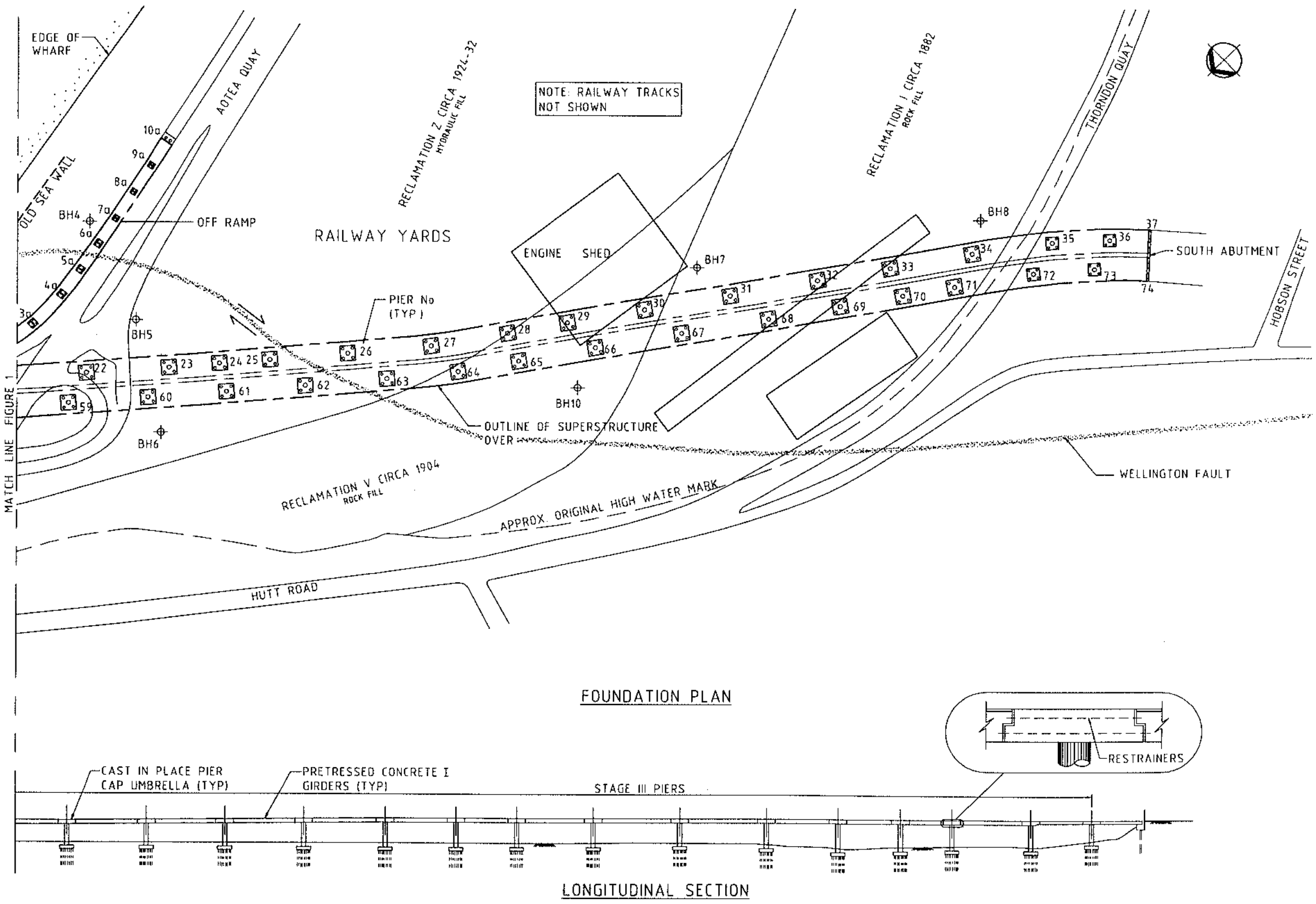
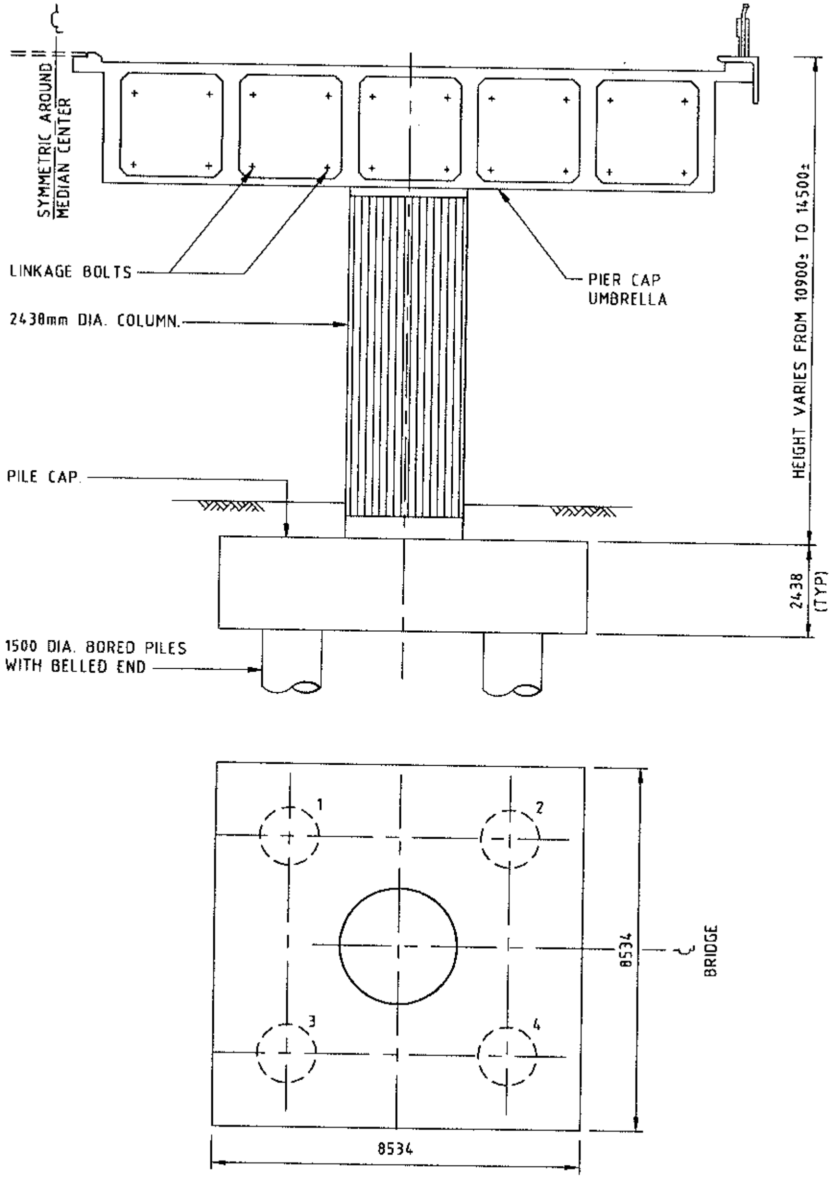
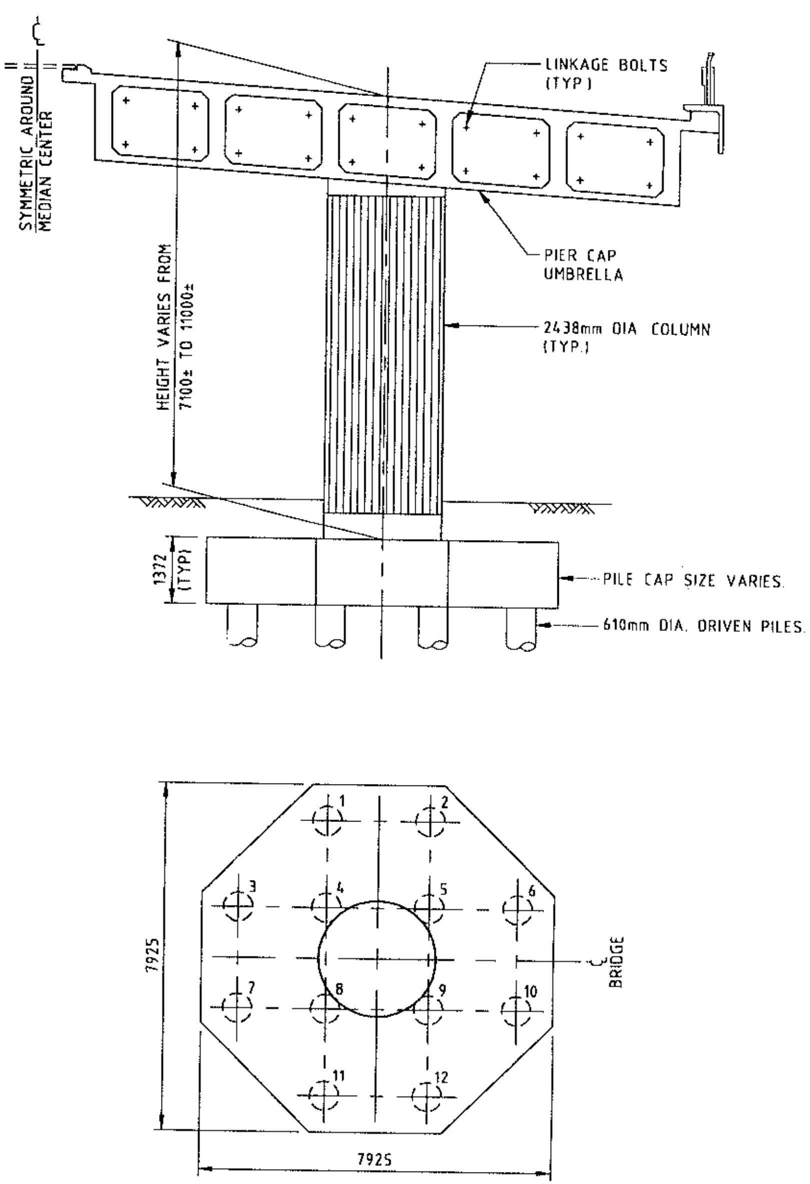


FIGURE 2 - THORNDON OVERBRIDGE

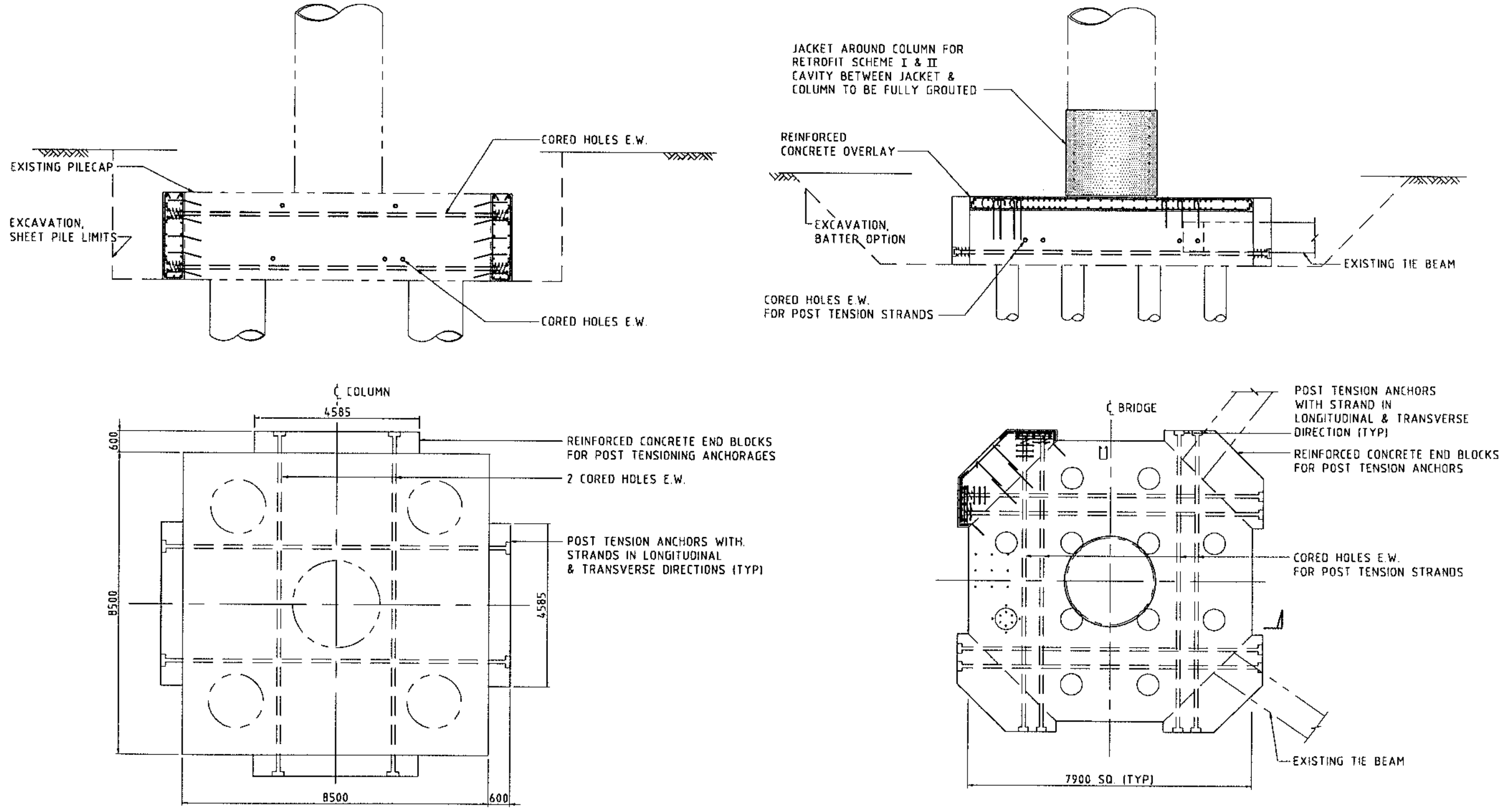


TYPICAL STAGE III PIER



TYPICAL STAGE II PIER

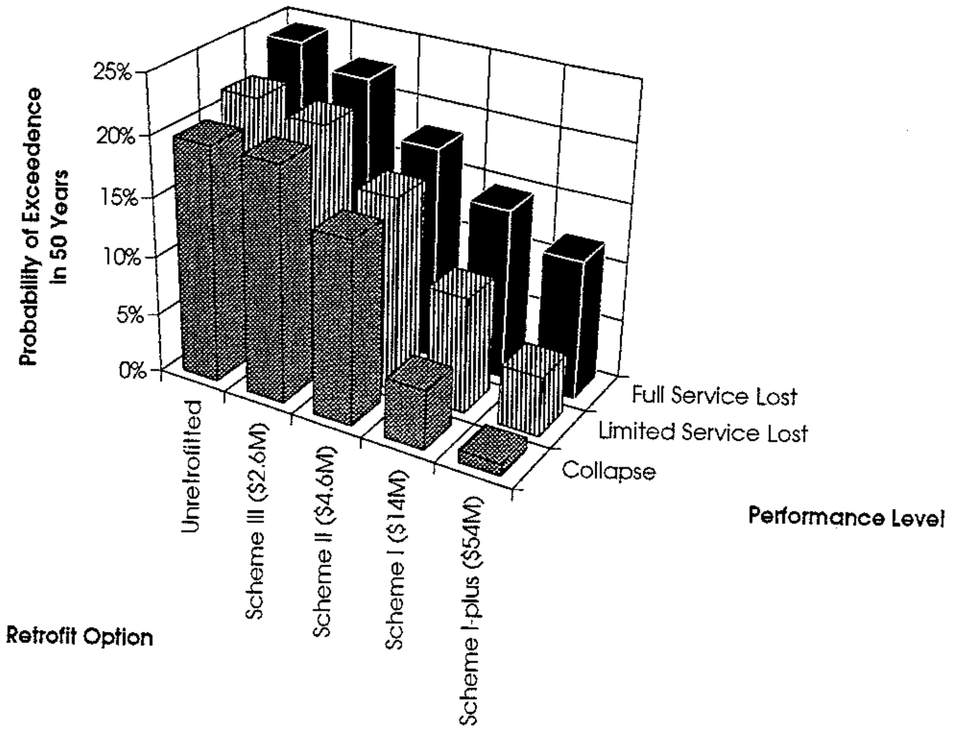
FIGURE 3 - THORNDON OVERBRIDGE EXISTING PIERS



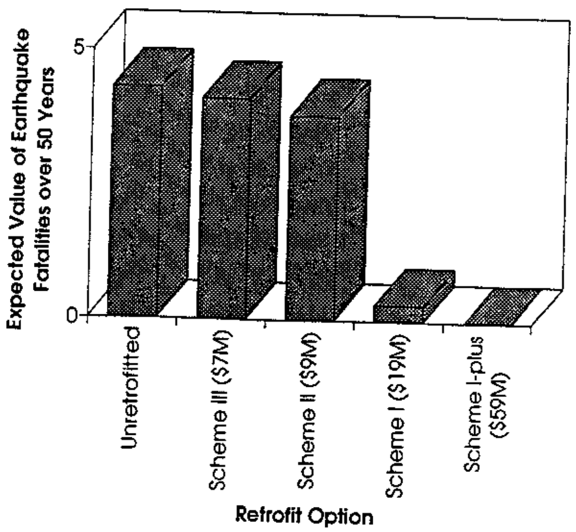
TYPICAL STAGE III PIER RETROFIT

TYPICAL STAGE II PIER RETROFIT

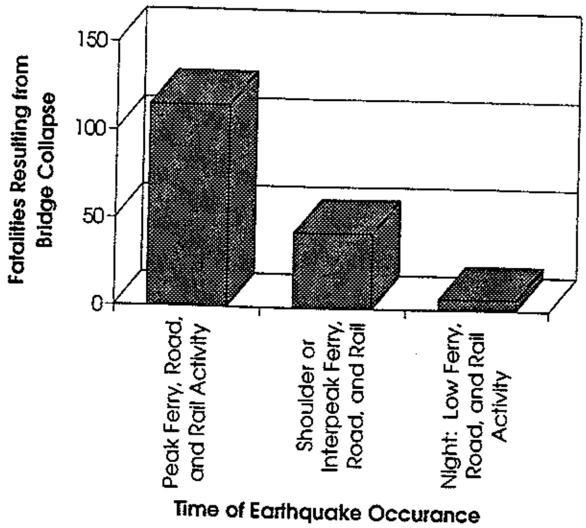
FIGURE 4 - THORNDON OVERBRIDGE PIER RETROFITS



OVERBRIDGE EXCLUDING OFF-RAMP



EXPECTED VALUE OF
EARTHQUAKE FATALITIES



ESTIMATED FATALITIES
FROM BRIDGE COLLAPSE

FIGURE 5 - THORNDON OVERBRIDGE PERFORMANCE DATA