

# Design and Construction of the New Upper Harbour Crossing

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## SYNOPSIS

The New Upper Harbour Bridge consists of a 458m long balanced cantilever bridge together with approach spans and is under construction alongside the existing crossing of the Waitemata Harbour in Auckland. The new bridge is being implemented under a Design and Construct contract.

The design employs a wide transversely post tensioned single cell box girder, elimination of joints and bearings (except at abutments), use of approach span deck units for temporary access staging, two stage pours for cantilever segments and the use of external and internal longitudinal post tensioning in the box girder. It meets or exceeds all project aesthetic, environmental, durability and operational requirements and was proved via the tender process to be particularly cost-effective.

## 1 INTRODUCTION

The New Upper Harbour Bridge crosses the upper reaches of Auckland's Waitemata Harbour adjacent to the existing two-lane Upper Harbour Bridge and approximately 10km up the harbour from the well-known Auckland Harbour Bridge. The bridge forms part of Transit New Zealand's Upper Harbour Corridor motorway project that will provide an east-west link between Auckland's North-Western and Northern motorways. The new bridge will carry the east-bound carriageway of the new motorway and the existing bridge will carry the west-bound carriageway. The 17.8m wide deck of the new bridge carries three traffic lanes and a footpath/cycleway.

A contract for the Design and Construction of the bridge and for Construction of the widening of the existing western approach causeway was awarded to Fletcher Construction in January 2003, with Beca Carter Hollings and Ferner Ltd as the Contractor's Designer.

The new bridge is expected to be completed in late 2005.

## 2 PROJECT REQUIREMENTS

### 2.1 Site Description

The new bridge is immediately adjacent to the existing two-lane concrete balanced cantilever bridge, which was designed and constructed in the early 1970's and is 457m long, with five 73m long main spans and two 46m long end spans. About 400m of the bridge (all but the eastern end

span) is over water with depths typically of the order of 8-10m at mid-tide and a deeper navigable channel passing under the eastern main span (refer Figure 1).

The bridge vertical alignment rises from its western (Hobsonville) abutment, situated at the end of an 850m long approach causeway constructed over a shallow tidal inlet, to the Greenhithe abutment 25m above mean sea level. The maximum gradient, over the eastern spans, is 5%. The horizontal alignment is straight throughout the bridge.

Broadly, ground conditions consist of recent marine sediments overlying Tauranga Group alluvium, which is in turn underlain by moderately weathered interbedded Waitemata Group siltstone and sandstone. The uppermost Waitemata Group layer is in some locations weathered to residual soils. At some piers the recent marine sediments directly overlie the Waitemata Group.

At the locations of Piers 1-3 of the existing bridge competent Waitemata Group rock is found within 1.5m of the seabed. At Piers 4 and 5 a gully has formed within the Waitemata Group rock that is infilled with very soft to soft Tauranga Group alluvium (typically SPT  $N < 1$ ), so that at these piers the competent rock is 10-11m below seabed level.

## **2.2 Principal's Requirements**

The Principal's Requirements for the project included the following constraints on the design and construction of the new bridge:

- The bridge should be located immediately to the north of the existing bridge, and follow the same vertical alignment
- The bridge should be 17.8m wide, allowing for three traffic lanes plus shoulders and a combined footpath/cycleway together with barriers
- The bridge should have the same pier positions and span layout as the existing bridge, and have the same depths and profile in elevation (4.3m at piers, 2.0m at midspan, parabolic soffit profile)
- A box girder cross-section should be used, so as to visually match the existing bridge
- The box girder should be structurally continuous over its full length (unlike the existing bridge which has several joints with ongoing maintenance requirements).

The Design and Construct portion of the project includes an 80m length to the west of the existing bridge west abutment. Over this length, the Principal's Requirements allow widening of the existing causeway (as constrained by a limit to filling over the sea bed set by the project resource consents), construction of approach spans or a combination of the two.

## **3 DESIGN SOLUTION**

### **3.1 Interactive Tendering Process**

The contract documentation for the new bridge and causeway widening was based on Transit New Zealand's Design and Construct pro-forma, which requires an interactive tendering process.

Thus during the 18 week tender period and as the tender design for the new bridge was developed, the Fletcher/Beca team (and other tenderers) met regularly with the Principal and his Engineer to ensure that the design was acceptable. The process required three formal project submissions to the client as well as a road safety audit submission. The design was also required to be approved by an Aesthetics panel. The process culminated in the submission of a Preliminary Conceptual Design Report and Drawings some weeks before tenders closed, to allow the Principal to “*confirm without prejudice that there are no objections to the proposals*” and to formally comment on the proposals.

## **3.2 Box Girder**

### ***3.2.1 Form and Materials For Box Girder***

The following options were considered in developing the design for the box girder:

- Post-tensioned Concrete Box –Twin cell vertical webs (as was indicated in the Specimen Design supplied by the Principal to tenderers)
- Post-tensioned Concrete Box – Single cell box with inclined struts propping the top slab mid span, of similar external profile to the Specimen Design
- Post-tensioned Concrete Box – Single cell box with inclined webs and with wider cantilevers
- Steel Box Girder

The selection of the preferred option was driven by cost, constructability and aesthetic considerations.

The steel box option dropped out at a very early stage as it was assessed that this option was unlikely to provide an economic solution due to initial and full life cycle costs.

It was decided that construction of all concrete box girder options would be by the cast-in-situ balanced cantilever method. This was more economic due to the available construction programme time and the plant availability and preferences of the constructor.

A sufficiently detailed analysis of the concrete box girder options was undertaken to allow comparative evaluation. The single cell box with inclined webs and wider cantilevers proved to be the clear winner.

Identified advantages of this option were as follows:

- Constructability – this option is the easiest to build due to significant simplification in travellers and formwork for a single box/two-web arrangement as compared with a twin box arrangement.
- Less concrete volume than other options – i.e. two minimum size webs over the mid span sections rather than 3 and a narrower bottom flange.
- No internal struts are required.
- Webs supported directly on piers resulting in smaller pier diaphragms.

- Aesthetics. The selected option is less bulky than the other options. The sloping webs provide a more aesthetically pleasing appearance.
- Improved inspection and maintenance access with single box.

The longer deck slab cantilevers required transverse prestressing of the deck - however the additional cost and complexity of this is more than outweighed by the advantages listed above. The box cross-section is shown in Figure 3.

### ***3.2.2 Prestressing Arrangement***

The bridge is being constructed in 4m long segments with either two or four No. 22/15.2 mm strand tendons anchored at each segment. A total of 24 cantilever tendons are provided to give adequate load balance of the cantilevers during construction and of the completed bridge in the long term.

In the five central spans, 20 or 22 No. 12/12.7mm strand continuity tendons are provided in the box bottom flange. In the end spans, only 12 continuity tendons are required. The continuity tendons are designed in conjunction with the external tendons to satisfy the Transit New Zealand's Bridge Manual requirement of zero tension for Group 1A (long term permanent stresses plus traffic and footway loads) serviceability loading. However the controlling load case has been determined to be Group 2A (with differential temperature stresses included). In order to cope economically with the stresses under this load case, a partial prestress design has been carried out to verify that steel stress ranges (200 MPa) and crack widths (0.3mm) are within the permissible limits.

Two No. 12/12.7 mm strand stitching tendons have been provided at the top flange of the closure segments and outer end of the end spans, to avoid tensile stresses under all serviceability load cases. For the construction condition, additional temporary top flange tendons are used in the end spans to support continuation of the cantilever erection sequence to the abutment, in conjunction with props at the cantilever ends.

To provide sufficient prestressing to cater for the high superimposed dead loads and design live loads for this bridge, it was found to be cost effective to use additional external tendons. This avoids the need to install significant additional numbers of cantilever and continuity tendons.

Four No. 22/15.2mm strand external tendons are utilised with cast in-situ deviators at the midspan closure segments, and steel tube deviator saddles at the piers.

Provision was made for future external prestress (as nominated in the Principal's Requirements) by constructing holes through the pier diaphragms.

The longitudinal prestressing is shown in Figure 4.

A transversely prestressed design has been adopted for the deck, with 4/12.7mm strand tendons at 400mm centres. A partially prestressed design approach (i.e. satisfying crack width and stress range requirements rather than tensile stress requirements) has been adopted. Group 1A loadings with allowable stress range of 100 MPa and a crack width of 0.1mm governed.

### 3.3 Substructure and Foundations

The selection of the preferred substructure solution was driven by cost, constructability and structural performance. Particular emphasis was given to developing a design that eliminated the need for bearings at the piers so as to reduce the resulting construction complexity and cost and future maintenance requirements. Piling arrangements have a significant influence on structural actions, therefore foundations and piers were considered jointly to produce an optimal design.

The following options were considered for the piers:

- Single reinforced concrete box column piers.
- Twin box column piers with twin slab wall piers for Piers 1 & 2 to accommodate superstructure shortening effects
- Twin box column piers in conjunction with a longitudinally soft pile arrangement (see first pile option below).

The following options were considered for the bored pile foundations:

- Two large piles per pier
- Four or more piles per pier at various pile spacings

The twin box column (2.4m x 2.4m) pier with two large (2.4m diameter bored piles) was found to be the optimum solution for the completed structure. However the resulting construction stage deflections due to out of balance segment loads were found to be excessive at Piers 4 and 5, where a considerable depth of Tauranga alluvium overlies the Waitemata material. At these piers a 4 pile (1.5m diameter) arrangement was adopted.

Discrete pile caps joining piles only with the column directly above are provided at Piers 1-3 and Pier 6. The discrete pile caps reduce the overall transverse stiffness of the foundation at these locations due to the elimination of framing action at the pile cap level. This results in an improved transverse seismic load share between the shorter piers (which have the discrete pile caps) and the longer piers 4 and 5 (where the piles are framed with larger pile caps linking all piles in the group).

Identified advantages of the substructure arrangement adopted are as follows:

- Least number of piles with significant reduction in set up effort required
- Smallest overall volume within temporary caissons resulting in significant savings
- Less materials input
- Significant reduction in pile cap size for most piers due to piles being located directly below columns
- Increased longitudinal flexibility for most piers due to elimination of framing action in this direction eliminating the need for pier bearings.

- Efficient vertical load carrying arrangement. Vertical loading on the single row of piles is unaffected by longitudinal loading effects as would be the case with the multiple row pile groups.
- Columns same width in elevation as those of the existing bridge

Reliance on only two piles to support each pier may be seen as offering less redundancy and robustness - however ground conditions are well understood in the vicinity of the bridge and are well suited to the type of construction proposed. The design requires jacking in the Span 2 and 5 closures to reduce the long term superstructure shortening effects on the outer piers which are more greatly affected by shrinkage and creep (Piers 1 & 6). However this is not thought to be a significant disadvantage.

In summary, the adopted column and pile arrangements for the main bridge have significant constructability, cost and structural performance advantages over all other options evaluated.

Figure 2 shows the adopted substructure arrangement (drawn at Pier 4).

### **3.4 West Approach Spans**

In the early stages of the tender design development considerable effort was invested in finding the right balance between approach spans versus retained/embankment type construction. At the end of the causeway the construction boundary does not provide sufficient width for traditional embankment construction and therefore either a retained embankment or bridge solution were the only viable options for this zone.

Options examined were:

1. Single approach span using prestressed concrete 'I' girders or 'Tee Roff' sections with remaining causeway retained and or traditional embankment construction.
2. Multiple span prestressed concrete 'I' girders or "Tee Roff" girders with remaining causeway retained and/or traditional embankment construction.
3. 1, 2 & 3 span Double Hollow Core prestressed concrete box beam spans with the remaining causeway retained and/or traditional embankment construction

The 3 span Double Hollow Core option stood out as being the optimum solution.

Advantages of this approach are as follows:

- Constructability – no soffit formwork is required as with the 'I' girders
- No in-situ top slab required
- No costly retaining walls. The three span option takes the bridge construction beyond the zone requiring retaining walls to remain within the construction boundary constraints.
- Simplified integral construction.

In order to address potential concerns about reflective cracking through the surfacing over the joints between the DHC units and to assist in the resistance of the large vehicle side protection

forces, a relatively high level of transverse prestressing ( 5 x 920kN UTS tendons per span) has been adopted.

Precast concrete drop panels are used along the northern edge of the deck to give a visible structural depth in keeping with the main bridge.

Figure 4 shows the approach span cross-section.

### **3.5 Outcome of Tendering Process**

Tender evaluation was carried out using Transit's Quality/Price tradeoff method. The Fletcher Construction bid was a clear winner. This was attributable to:

- The efficient bridge design
- A cost-effective alternative design developed by Fletcher and Beca for the causeway widening portion of the contract
- The quality of the methodology, relevant experience, technical and management skills able to be demonstrated by the Fletcher / Beca team in the tender submission.

### **3.6 Detailed Design**

Detailed design for the bridge was completed in 2003 in several packages to suit the Fletcher construction sequence and programme, with each package required to pass through an independent Checker and to obtain a Building Consent and advice of either "no objection" or "non conformance" from the Engineer.

The staged construction of the bridge incorporating time-dependent creep and shrinkage effects was modeled using the sophisticated Bridge Designer 2 software, enabling detailed calculation of stresses, jacking forces and deflections; both during construction and for the completed structure in the long term.

During detailed design several "value-adding" proposals were developed, offered to the Principal and accepted. These included the use of micro-silica concrete in the substructures and parts of the superstructure to give improved durability above that given by Bridge Manual requirements, and the provision of small amounts of additional reinforcement to give an improved level of seismic resistance above that specified.

## **4 CONSTRUCTION SOLUTIONS**

### **4.1 Access**

Access and provisioning to piling locations and subsequent bridge construction is via a mixture of heavy and light staging from the west approach causeway.

Heavy staging is provided locally to piers and approach spans and allows crawler and rough terrain crane operations in these locations. The heavy staging employs tube piles, steel crossheads, longitudinal 12m long I-beams, and a timber deck. Light staging provides vehicle

access only for up to 25T rough terrain cranes, concrete trucks and reinforcement delivery and serves as a link between the heavy staging areas at each pier. A significant innovation was the use of the approach span Double Hollow Core beams as part of the light staging deck. The staging uses a single tube pile, steel crosshead, and the approach span beams, employing temporary shear keys and transverse post-tensioning.

## **4.2 Foundation, Pilecap and Column Construction**

The main bridge bored piles require permanent casings, which are installed to above high tide level to allow all tide access for piling. The piling plant utilises a crane mounted drilling rig on a jack-up barge with pile gates cantilevered off the side of the barge.

As the pile caps are completely submerged at all tides a steel cofferdam is employed with a sacrificial floor and removable walls, which also act as the formwork for the pile cap pours. The overlength pile casings are utilised temporarily to support the cofferdam for buoyancy and gravity loads whilst the coffer dam floor seals are completed, and permanent welding of the floor to the pile casing completed. The surplus length of casing is then cut off.

The piers columns, which range from 6.5m to 19m tall, are poured with independent internal and external 7m jump forms.

## **4.3 Box Girder**

The 7.2m long pier table is supported by brackets stressed onto the pier columns, supporting steelwork, and formwork.

Box girder construction is in 4m segments, each constructed with a bottom slab and web pour and separate top slab pour.

Although there are some disadvantages of pouring each segment in two pours rather than one, two pours were utilised for the following reasons:

- Greatly simplified internal web formwork
- Greatly simplified internal top slab formwork
- Traveller design is required to cope with only partial segment loads
- Ability to 'resource level' carpentry and subcontract crews within each pair of travellers
- Reduction or elimination of requirement to kentledge bridge at outer cantilever extremes
- Better access for bottom slab construction.

Box girder end segments at the abutments will be constructed supported on the abutment crosshead and temporarily propped to allow the last cantilever segment to be 'closed' to it.

## **5 CONCLUSION**

This paper has briefly described the features of a significant harbour crossing which incorporates a number of innovative design and construction features. These features together with the close

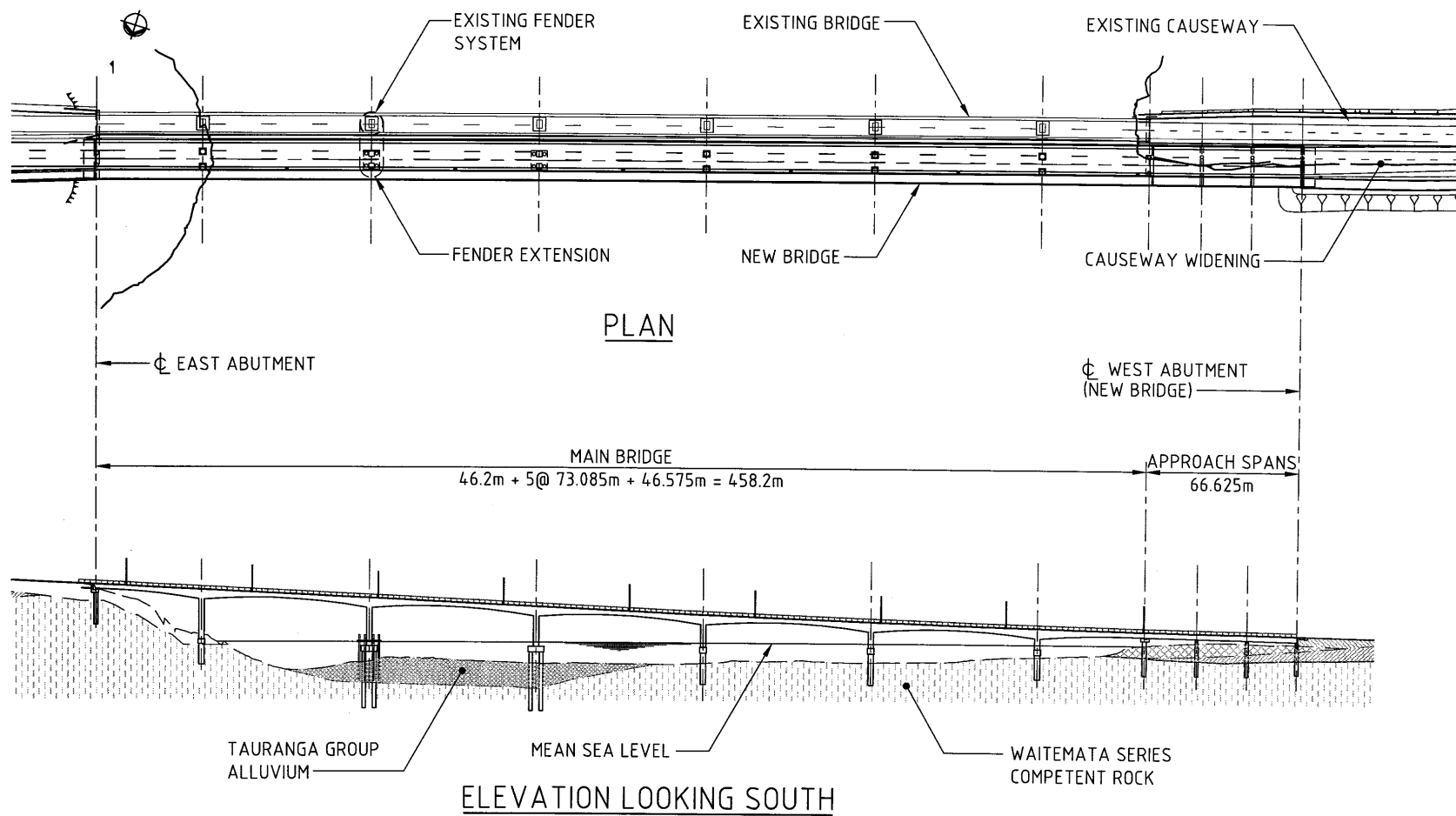


teamwork which has characterised the project have resulted in a particularly cost effective design which meets or exceeds the project requirements.

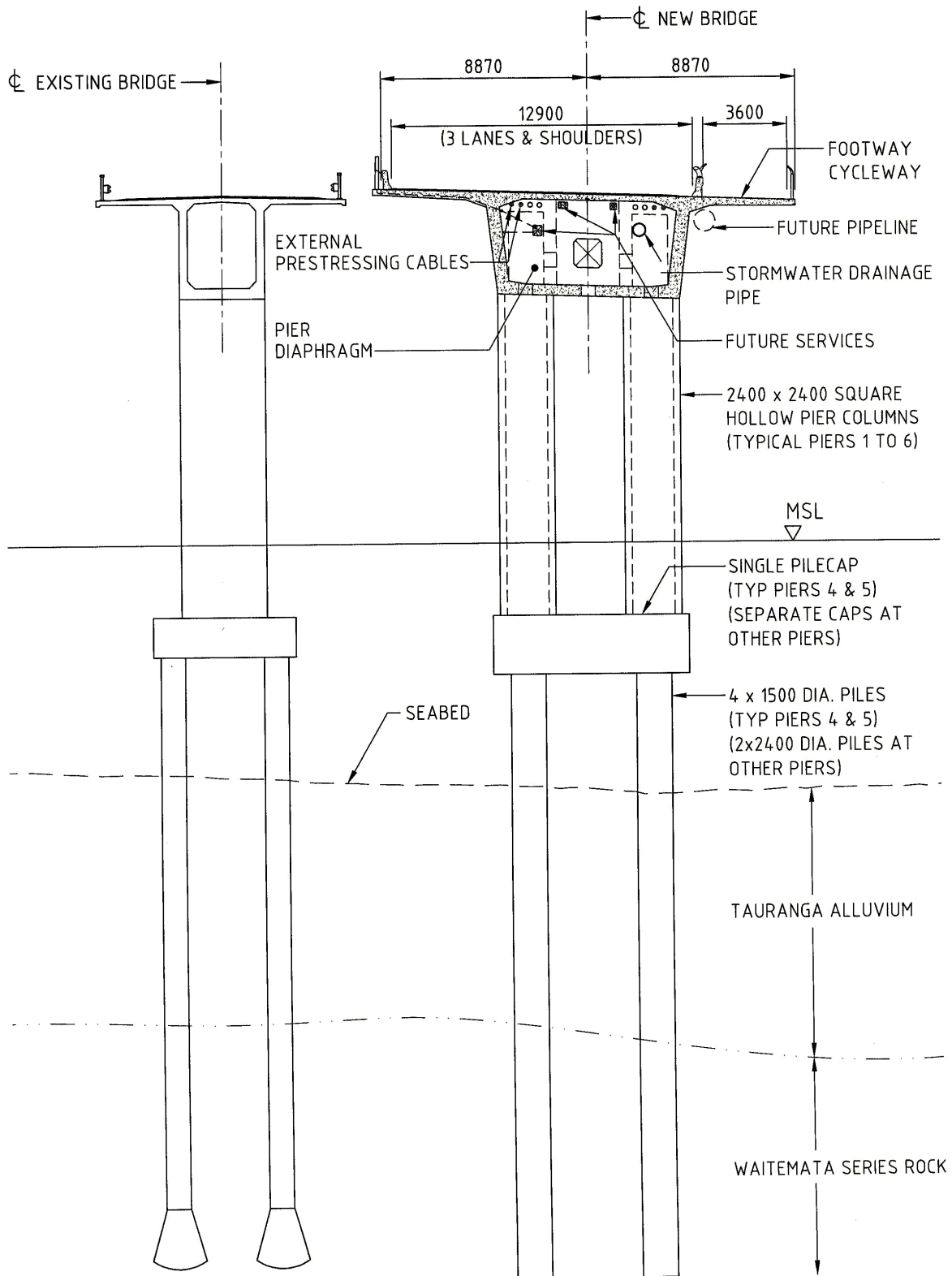
## **ACKNOWLEDGEMENTS**

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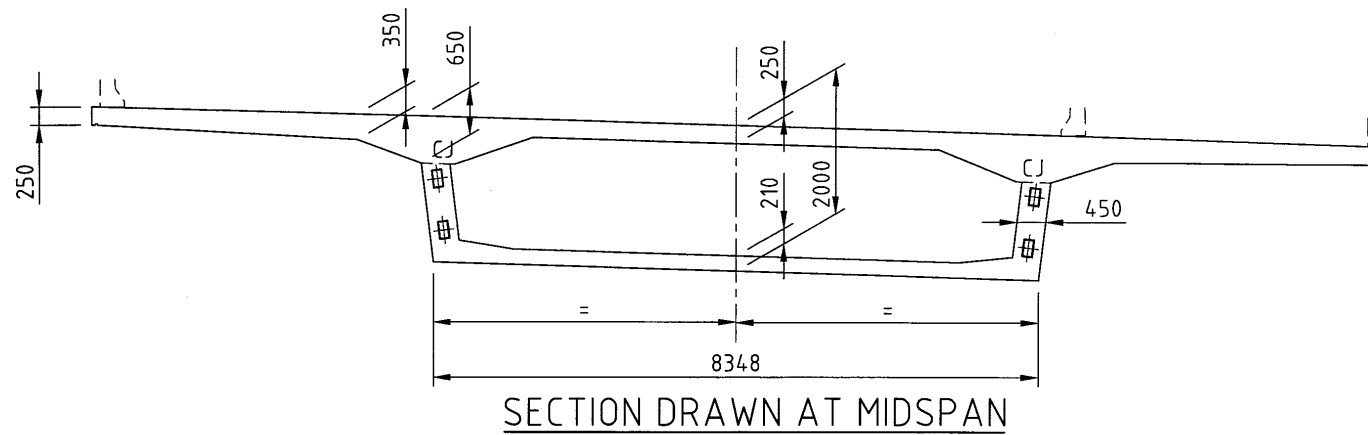
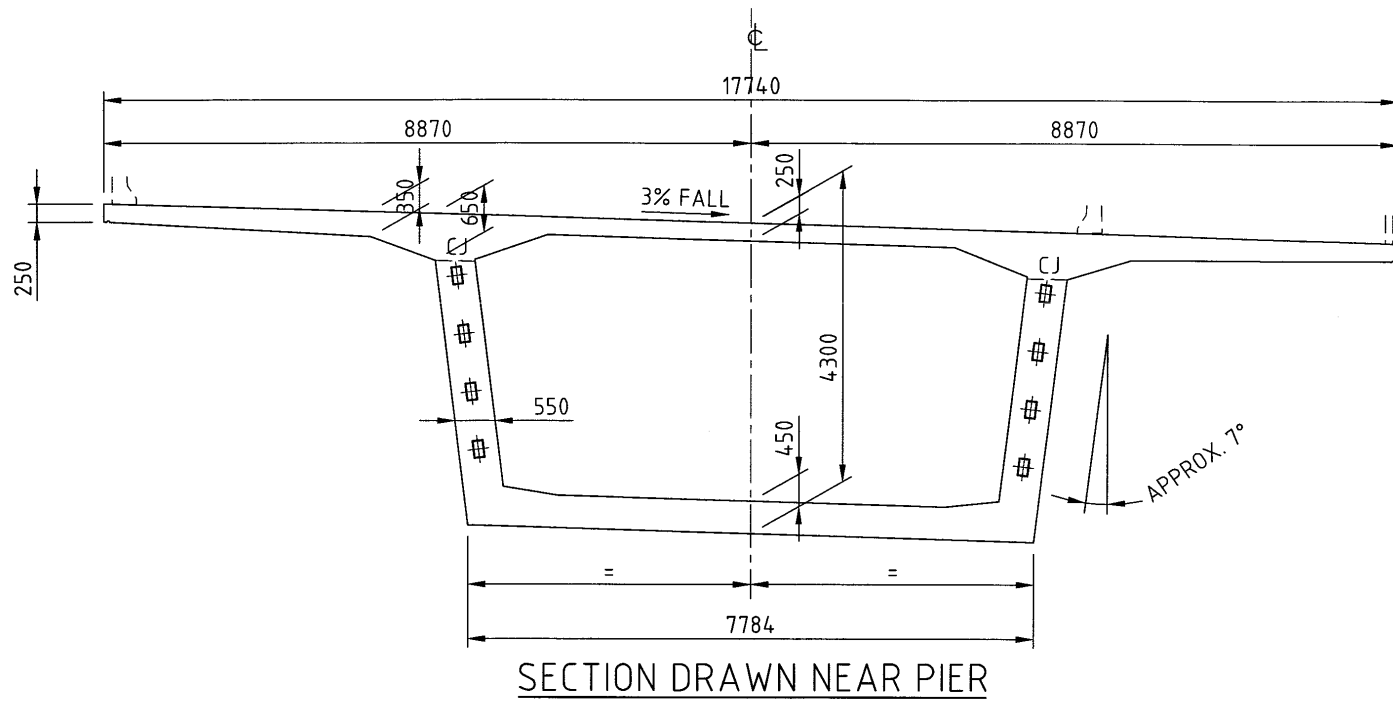


*Figure 1: Plan and Elevation*

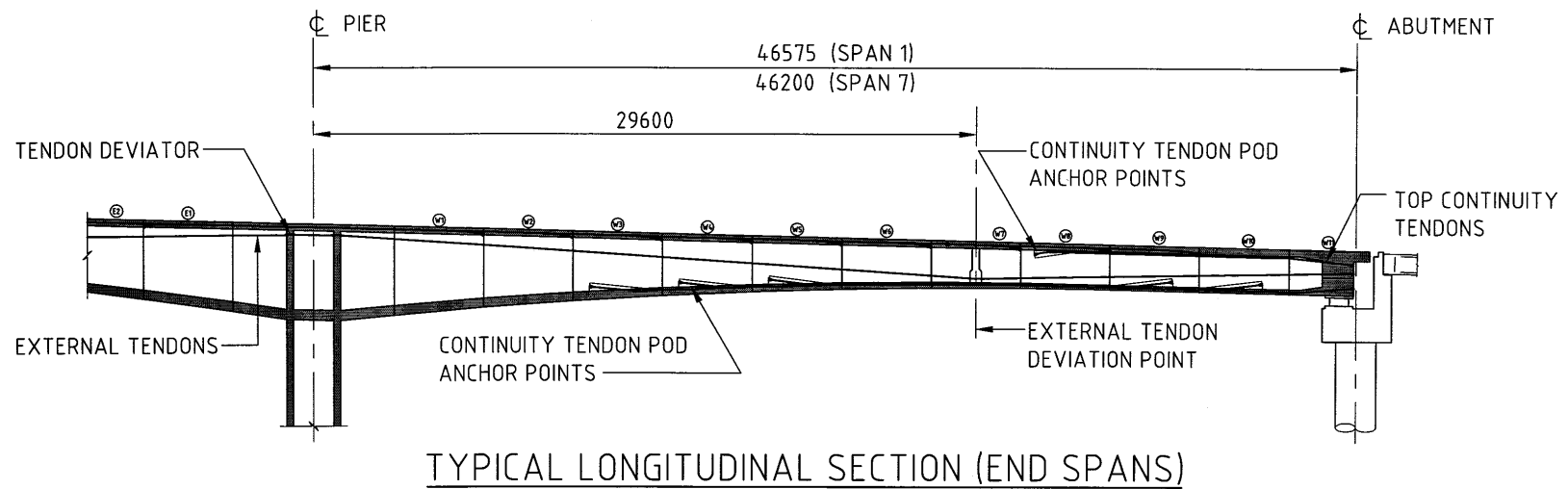
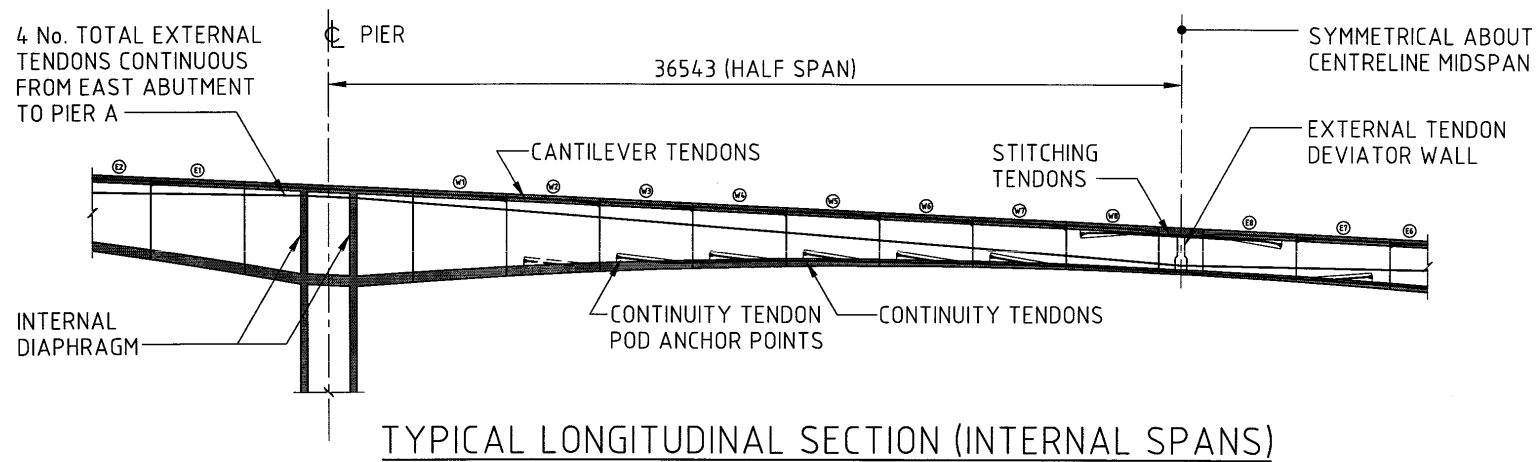


PIER 4 LOOKING WEST

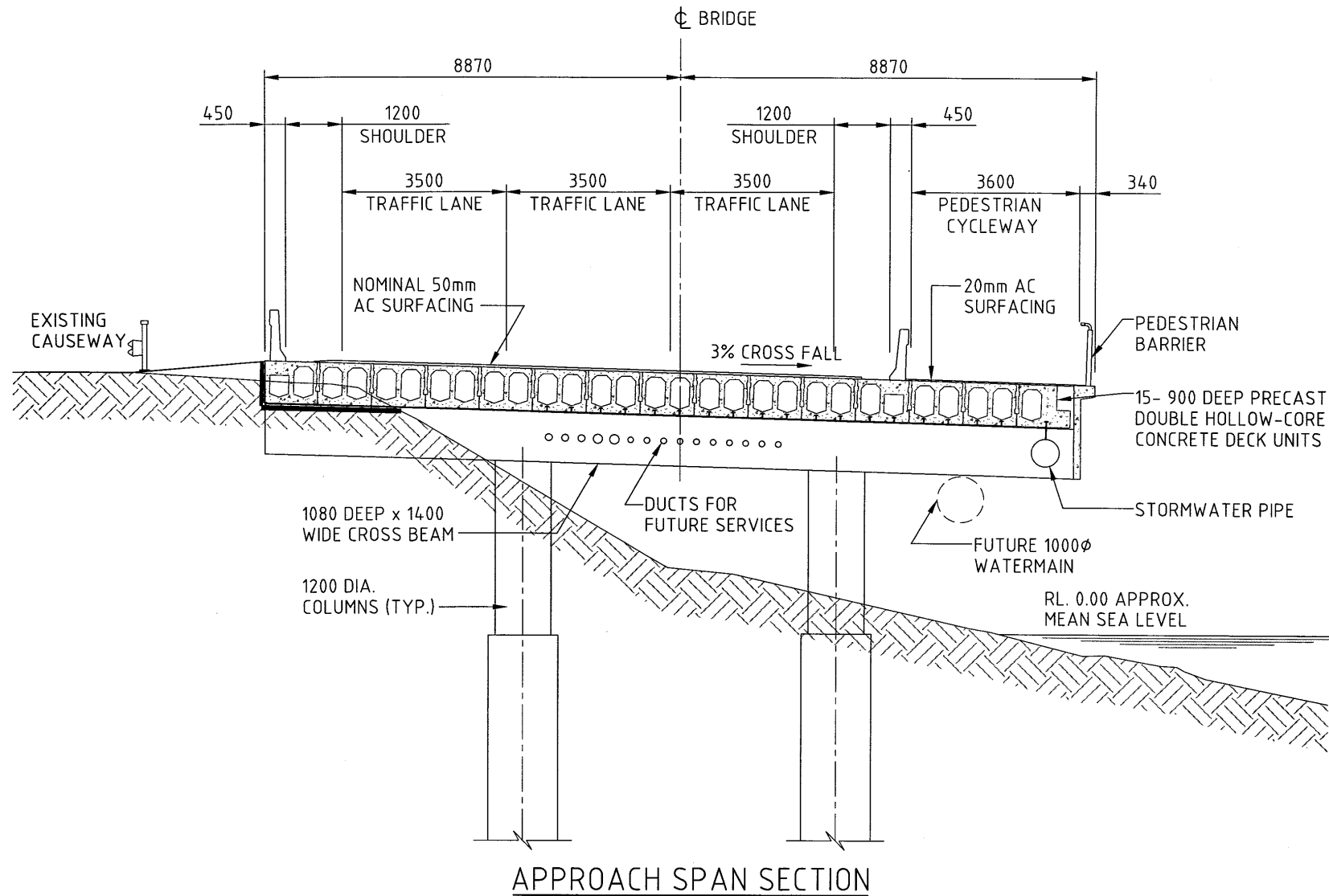
**Figure 2: Main Bridge Cross-Section**



**Figure 3: Box Girder Dimensions (Pier and Midspan)**



**Figure 4: Box Girder Longitudinal Prestressing**



**Figure 5: Approach Span Cross-Section**