The New (Duplicate) Mangere Bridge is a 646m long, 21.5m wide motorway bridge currently being constructed using balanced cantilever techniques alongside the existing crossing of the Manukau Harbour between Mangere and Onehunga on State Highway 20, Auckland.

The bridge design and construction is carried out for the New Zealand Transport Agency (NZTA) by the Manukau Harbour Crossing Alliance (MHX – Fletcher Construction, Beca Infrastructure, Higgins and NZTA) after the alliance was successful in a competitive TOC (Target Outturn Cost) process involving two alliance teams.

The bridge design employs twin (linked) cast-in-situ box girders with 100m main spans, the superstructure being continuous from abutment to abutment, and supported on flexible piers which allows the elimination of bearings on the main piers.

Particular attention is given in the design and during construction to achieving the desired whole-of-life performance and durability of the completed structure

This paper describes the background and rationale behind the design concepts implemented in the final design and the construction techniques being used to achieve completion by the required 2011 completion date.

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INTRODUCTION

The New Mangere Bridge crosses the upper reaches of Manukau Harbour adjacent to the existing 4 lane bridge designed and constructed in the 1970’s. The new bridge is a key component of NZTA’s Manukau Harbour Crossing Project\(^5\), aimed at improving traffic flow between Auckland International Airport and Auckland City and will allow 4 traffic lanes plus a 4m wide bus priority shoulder in each direction across the Manukau Harbour. Southbound traffic will in future use the new bridge and northbound traffic will be carried by the old bridge.

The bridge and wider project are required to be completed in early 2011 in time for the Rugby World Cup.

PROJECT REQUIREMENTS

Site Description

The new bridge is situated to the east of the existing 644m long twin box balanced cantilever bridge with a gap in-between of about 2m. The design of the existing bridge allowed for a duplicate bridge to be built alongside, the two bridges to be supported on shared abutment structures. The duplicate bridge’s portion of the north abutment was built as part of the original contract but only a portion of the duplicate bridge’s south abutment was built at that time.

Approximately 435m of the new bridge is over water with a maximum depth at MHWS of about 13m in the designated 45m wide navigation channel. Much of the remaining length is built over previously reclaimed land at both ends of the bridge.

At the northern end the bridge also crosses Onehunga Harbour Road and Onehunga Branch Line.

The founding conditions consist of extremely to very weak (SPT N 25 to 100) East Coast Bays Formation sandstone (ECBF) overlain in order of increasing depth by:

- 2-4m of recent marine sediment (except at both ends of the bridge where the marine sediment has been removed and replaced with granular fill)
- 0-7m of Tuff (Auckland Volcanic Field)
- 6-11m of Tauranga Group Alluvium

\(^5\) The Manukau Harbour Crossing Project includes 4km of motorway widening to give 3 lanes plus 4m bus priority shoulder in each direction and replacement or widening and/or lengthening of 5 minor bridges.

Bridge Geometry

The geometry of the new bridge is similar to that of the existing bridge with approximately 1/3\(^{rd}\) of the length on a straight and 2/3\(^{rd}\) on a 902m radius horizontal curve. Most of the bridge is situated on a 8200m radius crest curve with slopes varying from 2.7% at the south end to 3.9% at the north end.

Superelevation is 3% to the east at the south end and 5% to the west at the north end.

NZTA Requirements

NZTA’s requirements included the following constraints on the design and construction:

- Must provide for two 45m wide shipping channels in locations defined on the existing bridge drawings - a navigation channel with 12.2m minimum vertical clearance above MHWS and a berthing channel with 11.8m minimum vertical clearance above MHWS.
- Pilecap levels must match those of the existing bridge. No more than 6 piers may be in the water.
- New bridge to be either prestressed concrete box girder or steel box girder with concrete deck. Girder to be of variable depth with minimum internal headroom of 1800mm. Aesthetics to be “sympathetic” to the architectural styling of the existing bridge.
- Clear width to include four 3.5m lanes and one 4m left-hand shoulder.
- Bridge to be continuous between abutments. Deck joints to be finger type.
- Design life to be 100 years. All combinations of concrete mix and cover to reinforcement to be demonstrated by chloride-based modelling to give 100 year life in the environment they will exist in.
- All post-tensioning components to have three layers of protection against ingress of corrosive elements to strand. Prestressing duct to be plastic to FIB 7 and coupled across construction joints, anchors to have permanent grout caps.
- Allowance to be made for future installation of a 1.6m diameter watermain.

After award, NZTA’s requirements were amended to allow for a possible future rail bridge to share a portion of the new bridge alignment. Provision has therefore been made for a low level single track rail bridge to be constructed between the two columns of some of the new bridge piers, simply supported by the new pilecaps.
DESIGN SOLUTION

Preliminary Concepts

As part of the TOC process, a team of senior cost, planning, design and construction engineers was assembled to conceive and then evaluate several options for the new bridge. Options considered included conforming and non-conforming concepts to ensure that ideas were not prematurely abandoned and preliminary designs for each were advanced sufficiently for preliminary planning and cost estimation purposes. Some of the options considered were:

- Constant depth steel triple box girder with concrete deck and 98m spans,
- Constant depth steel I-girder bridge with concrete deck and typically 51m spans,
- Constant depth incrementally launched single cell prestressed concrete box girder with typically 65m spans,
- Variable depth in-situ balanced cantilever bridge with twin cell and twin box options, the latter with twin leaf columns and 90m + 10m span arrangement or single column option with 100m spans.

The preferred solution was selected based on consenting issues, cost, constructability and aesthetic considerations, the steel box and incrementally launched concrete box options falling out early based on cost and consenting issues. The decision to proceed with the single pier 100m span balanced cantilever option over the 90m + 10m span twin leaf pier alternative was based on constructability, consenting and program issues and the decision to adopt twin boxes rather than a twin cell box was based on cost.

Design of the preferred solution was further advanced for detailed planning and costing purposes and submitted together with design for the remainder of the project and the TOC for evaluation following which the project was awarded to the MHX Alliance.

Box Girder Form and Materials

The preferred superstructure solution as adopted for final design is as follows:

- Box depth varies parabolically from 5.5m at the piers to 2.75m at mid-span in the typical spans and is constant 2.75m deep in the end spans.
- Webs in the typical spans are constant 300mm thick and in the end spans vary between 550mm and 300mm.
- Soffit thickness varies parabolically from 700mm at the piers to 230mm at mid-span in the typical spans and between 430mm and 230mm in the end-spans.
- Deck is typically 240mm thick with 500mm deep haunches at the webs.
- Superstructure concrete is 50MPa using only GP cement, with 50mm external cover (demonstrated by modelling to give 100 year life, based on surface chloride levels measured on existing bridge and curing with curing compound (refer paper ‘Manukau Harbour Crossing Durability Modelling’ by Cook and Dickson)
- Greywacke aggregate is used in all concrete. For the superstructure concrete, this needed careful consideration of the likely creep and shrinkage of a greywacke aggregate concrete, which is known to be higher than those of the basalt aggregate concrete alternative which was also considered. Testing to date has confirmed that the performance of the greywacke concrete selected is consistent with the RRU70 based design assumptions (refer paper by Lipscombe and Dickson at Austroads Bridge Conference 2009). The lower density of the greywacke is advantageous from a pot tensioning quantum viewpoint.
- Segments are typically 4.1m long with 2m span closure segments and 7.8m pier tables.
- Cantilever tendons (tendons in the top flange arranged in a herringbone fashion from one cantilever tip to the opposite tip) are anchored in the upper haunches on each side of the webs so up to 8 tendons can be anchored in each segment.
- Continuity tendons (tendons in the bottom flange near mid-span) are anchored in blisters constructed on top of the bottom flange.

Figures 1 to 4 show the general arrangement of the bridge and typical pier, girder and prestress arrangements.

Prestressing Arrangement and Detailing

The final design requires 408 strands of 15.2mm EHT prestressing strand per box in the top flange at the piers (cantilever tendons). One of the
advantages of adopting a twin box rather than a single cell solution is that there are 4 pairs of web haunches rather than 3 in which to anchor the tendons which allows a greater number of smaller diameter tendons to be used - in this case 34 12-strand cantilever tendons are provided per box. The smaller tendon size allows reduced deck thickness and smaller haunches (to accommodate the anchors) with significant weight reductions and less prestress required to balance dead load. The use of 4 webs also reduces the deck spans so that the deck thickness reduction made possible by the use of smaller tendons can be realised.

The continuity prestressing also uses 12-strand 15.2mm EHT tendons so that the same materials and prestressing equipment can be used for cantilever and continuity tendons. Up to 14 tendons per box, supplemented with non-prestressed reinforcing, are required at mid-span in the 100m spans, the design using a partially prestressed solution with TNZBM Group 2A being the governing serviceability load case.

No external tendons are used in the design since this would involve an extra critical path activity at the end on the bridge construction and was also found to have little relative benefit compared to standard continuity tendons. However, provision has been made for future installation of two 22 strand external tendons in each box in compliance with the Principal’s Requirements.

All ducts are 80mm corrugated plastic with proprietary plastic couplers and plastic grout caps, detailed to be water-tight. Grouting is carried out in accordance with the Concrete Institute of Australia Recommended Practice (2007), with a low bleed grout (<0.5% in vertical cylinder with single strand acting as a ‘wick’).

See Figure 4 for typical prestressing layout.

Substructure and Foundations

The portions of existing abutment structures built on the new bridge alignment are inappropriate for the new bridge design and new independent abutments will be built in front of the existing structures - at the south end this requires partial demolition of the existing abutment but at the north end the existing structure will be retained behind the new abutment. The new abutments each have two piles connected by capping beam and support the bridge ends on sliding bearings. Concrete and steel shear keys provide transverse restraint to the ends of the bridge.

The bridge piers all have two rectangular columns, 2mx3m with a flare at the top for mainly aesthetic reasons. On the typical piers (2 to 6), the columns connect monolithically with the superstructure via a diaphragm that also provides transverse framing action.

The two end piers (1 and 7) are similar in appearance but support the superstructure on elastomeric bearings, two per box, that effectively provide a moment connection to each box in the transverse direction and a pinned connection in the longitudinal direction. Importantly these bearings also reduce the longitudinal movements that the piers would be subject to from creep, shrinkage, temperature and seismic effects.

The bearings were designed to achieve low total pier stiffness so that these shorter end piers will not attract excessive load during an earthquake. Each bearing is made of two 600 wide, 750 long and 337mm deep bearings placed side by side and linked together top and bottom by steel plates.

In order to further reduce the eventual shortening effects in piers 1 and 7 and also 2 and 6, the outer two piers at both ends will be jacked outwards by up to 42mm before making span 3 and 6 closures to partially offset long-term shortening effects, estimated to be 130mm at the end piers.

Typical pier columns are supported on single 2.4m diameter piles, connected by a 4m deep collar or cap, the top of which is at RL 0.2 and larger than the column on all sides by 0.5m. The end pier columns are supported on single 1.8m diameter piles, joined directly at existing ground level (RL 3).

Use of single large diameter piles rather than groups of smaller piles connected by pilecaps was chosen because this speeded up construction, reduced material quantities and reduced the size of the cofferdam needed to build the pilecaps. It also helped reduce pier stiffness to the extent that bearings are not needed on most piers and seismic or long-term shortening demands are not the governing design case for the piers or piles.

The piles are permanently cased from underside of pilecap to just into the East Coast Bays Formation (typically about 17m) with an uncased rock socket length below that of typically 24m. The piles

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6 The original PR’s required zero tension design but a departure was sought and granted that allows Partial Prestressed design because this is an in-situ segmental bridge with auxiliary non-prestressed reinforcing crossing the construction joints.

7 TNZBM group 2A load includes Permanent Loads + Temperature + 1.35xLive Load; for this project the 1.35 LL factor was replaced with 1.0

8 Because of the flexibility of the piers, unbalanced cantilever loads arising during construction would lead to excessive P-Δ effects if the piers were not braced. Temporary props will therefore be installed under the 4th segment from the pier to prevent the construction stage loads excessively governing pier and foundation design.
vertical loads are carried by a combination of skin friction and end-bearing, the design capacity of which was determined from geotechnical analysis and a 1.8m diameter pile load test conducted on site before detailed design commenced. The pile load test is discussed in more detail later in this paper.

The pile caps and columns utilise a 40% flyash 50 MPa concrete with 75mm cover, cured by leaving the forms in place for 7 days. This solution was demonstrated by modelling to give adequate durability (refer Cook and Dickson paper).

CONSTRUCTION TECHNIQUES AND SOLUTIONS

Construction Access

Access for piling and subsequent superstructure construction is by a 6.7m wide temporary jetty installed on the eastern side of the new bridge. Two “finger” piers extend out from this at 90 degrees and straddle each bridge pier to provide working area and access for constructing the substructure, erection of the form travellers and constructing the pier tables.

The temporary jetty employs 710 diameter steel tubes typically driven up to 18m below sea bed into the underlying ECBF by 6 tonne drop hammer. A steel frame commonly known as a ‘gate’ is cantilevered off the nose of the previous jetty section enabling accurate positioning of the piles. Transverse ‘crosshead’ beams rest on top of the piles and provide support for the longitudinal steel beams that support the 220mm thick precast concrete deck panels, capable of supporting a 100 tonne crawler crane with maximum payload. Production rates of 30m per day were achieved.

Test pile

The Alliance constructed a 1.8m diameter 40m long test pile 6m north of Pier 1 on the centreline of the new bridge with a 19m socket into the underlying extremely weak / very weak sandstone (ECBF).

The test pile was constructed using construction methodology consistent to that used for constructing the permanent piles, that is the pile was bored through a 21m long permanent steel casing installed through the overlying material and upon reaching pile founding level, the pile bore was left open overnight then flooded with water and the base cleaned out using a suction bailer. The time between completing excavation and commencement of the concreting was approximately 42 hours with concrete placed under water using tremie technique.

The pile was reinforced in two sections with a single 870mm Osterberg-cell placed 6.0m from the base of the pile, this location chosen so that the anticipated upward and downward load would be equally shared between the frictional shaft resistance above the O-cell and the frictional shaft resistance and contribution from base resistance below the O-cell.

An incremental load test as described in AS 2159:1995 was carried out followed by an additional extended proof load test. These tests showed that negligible shaft resistance was generated in the cased section of the pile and therefore pile design does not consider shaft resistance from this portion. The test pile load/deflection data for the portions of socket above and below the O-cell were then interpreted and used to determine acceptable factored design loads for the bridge piles so that pile settlement will be limited to about 15mm at all piers.

Bridge Piles

The bored piles for the bridge utilise permanent steel casings driven approximately 1m into the ECBF and the top of the casing cut off above the high water spring tide mark to prevent flooding.

The drilling rig then bores a shaft through the casing and into the ECBF to founding level (about 50m below the top of the casing) using an auger. The rock socket may only be exposed for 48 hours to prevent weakening of the socket interface. The reinforcing cage is pre-fabricated in 3 sections then lowered into place and spliced in-situ. Four sealed steel tubes for the purpose of sonic logging are tied inside of the cage and extend the full length of the pile.

Accurate orientation of the pile cage is necessary to ensure that the pile starters do not clash with the pilecap cage and is achieved by welding steel guide rails to the inside of the casing. Prior to pouring concrete, the entire cage is lifted out and the base of the shaft cleaned to remove debris.

A typical pile requires 190m3 of concrete which is placed by tremie tube embedded not less than 6m into the concrete at all times. A sonic logging test is performed after 7 days curing to establish that there are no voids and a further test performed after 28 days to verify that there are no ‘soft spots’ in the pile. Installation of two piles typically took 3 weeks.

Pilecaps

In accordance with the Principal’s Requirements, pilecaps are situated below mean sea level and steel cofferdams were therefore necessary to
provide construction access to the pilecaps that would otherwise be submerged most of the time.

These box-like dams were pre-assembled on land, lifted into place over the piles and welded onto the pile casing using 4 temporary props. The annulus between the dam and casing was then sealed with a special “anti-washout” concrete placed by divers using a tremie pipe. An equalising valve ensured that there was not a large differential in water level inside and outside the dam due to tidal variation until the concrete had set. The dam was then pumped dry, the props replaced with steel plates welded around the annulus, the excess pile casing cut off to expose the pile reinforcing and a false timber floor installed to provide a dry raised working platform to allow pumps to operate in case of dam leakage.

The cap reinforcing cage was partially pre-assembled (bottom mat and four side faces) on land using a template so that the vertical reinforcing positions would suit the pile reinforcing arrangement, allowing the cap cage to be installed over the pile cage without hindrance. The internal and top layers were then tied in-situ. The column kicker position was surveyed to precisely control the level of the column cage and position of the column. The cofferdam was kept dry at all times until curing was complete. Completion of a pile cap typically took 3 weeks.

Columns

As noted previously, chloride based testing indicated that leaving forms in place for 7 days would provide the required degree of curing in conjunction with 75mm cover and 40% flyash concrete. This modelling also showed that the required degree of curing would be achieved after 5800 degree-hours had accumulated (area under concrete temperature vs time graph). The earliest time at which the form could be stripped was therefore taken to be after 5800 degree-hours had accumulated, determined by measuring the in-situ concrete temperature using probes installed in the columns and connected to a data logger. To reduce heat loss through the form, a 19mm sealed plywood form was used.

The columns were poured full height in a single lift to avoid construction joints and save time, the latter being particularly significant because of the need to retain the form in place until adequate cure was achieved. It is estimated that it would have taken an extra 4 weeks to construct each column had 4m lifts been used. This did however require stronger and more expensive formwork because of greater hydrostatic pressure.

The reinforcing cage for the column excluding flare reinforcing was prefabricated on land then carried out onto the staging and placed onto the pilecaps by crane. The column formwork including the sides of the flares was installed as a single element and the flare reinforcing then fixed in-situ. The end faces of the flare were then shuttered and the column poured. Completion of two columns typically took 3 weeks.

Pier Tables

The pier table consists of a 2m solid concrete diaphragm linking the two columns and the first 2.9m long box segment on each side of both columns. Because of significant programme benefits this was constructed using a combination of precast and in-situ techniques rather than constructing it all in-situ (as is usual) on temporary brackets supported off the tops of the columns.

First the webs of the 2.9m long segments were individually constructed on a casting bed set up on site. All 8 webs in the pier table were then lifted into position on a ground mounted platform set up to simulate the longitudinal and transverse slopes particular to each pier and propped in position. The soffit slabs of all four segments were then poured between the web panels and a removable deck form then placed at the top of the web panels and the deck with construction joints at the interface with the diaphragm and at a deck stitch pour on centreline of the bridge. This created four individual box segments that could then be installed on top of the columns, the process typically taking four weeks. The segments were then transported individually onto the staging using a 6 axle commetto trailer, lifted by a 300t crane with two outriggers supported on the pile cap and two outriggers on two temporary 711mm diameter tube piles and installed on temporary brackets stressed onto the columns. The installation of the four units took approximately one day.
The temporary brackets also supported falsework to provide sufficient working area to install the diaphragm formwork and reinforcing in-situ. Installation of the diaphragm formwork and reinforcing, pouring the 160m³ diaphragm and striking of the formwork and falsework typically took about 5 weeks to complete.

Because of the size of the diaphragm, a 40% fly-ash concrete was used to limit heat of hydration. The forms were struck after 2000 degree-hours had accumulated (again determined to be adequate by chloride based durability modelling and measured by temperature probes installed in the concrete and attached to a data logger).

Both boxes on one side of the pier are constructed simultaneously with the opposite cantilever allowed to be up to one segment out of balance."¹² The form traveller is advanced onto a new segment when the concrete strength reaches 15MPa and before stressing the cantilever tendons in that segment, stressing following when the concrete reaches at least 25MPa at the jacking end and at least 20MPa at the non-jacking end. Temperature matched curing is used to determine realistic in-situ strength. Typically one pair of segments (comprising two box sections and linking deck slab each side of the pier) takes 8 days to complete. Surface curing is carried out by applying curing compound to all surfaces.

The temporary superstructure props comprise of a 711mm diameter tube underneath each web driven to set and depth to achieve flexural rigidity (buckling), compressive capacity and tensile capacity. A 610mm diameter tube is placed inside each of these piles with a cross head connecting the two 610mm props adjacent to each other. The 610mm tube is then raised up once the cantilever passes overhead and the two tube sections welded together to form a continuous prop. The crosshead is also tied to the soffit slab to transfer tensile forces into the box. Installation takes 2 days.

The 2m closure pours are completed in a prescribed sequence using separate temporary works (not form travellers). Barriers are completed instu in a conventional manner.

End Spans

End spans are constructed using a combination of in-situ and precast construction. The typical end span segments are cast in the same manner as the pier table precast units except the segment length is 3.5m and 6 units are cast in one operation (rather than four 2.9m units as for the pier tables). The solid diaphragms for the abutments and piers are also precast, where

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¹² Segments are constructed in two stages until a temporary prop is installed below one cantilever to limit the out-of-balance loads where after segments are constructed in a single stage.
access is easier and temporary works can be minimised.

The end span “pier tables” are constructed in two 10m long precast sections stitched together on a bed adjacent to their final position (one per box, each including the pier diaphragm and two 4.1m long segments on the same side of the pier). The complete 10m element is erected with the diaphragm supported on the permanent elastomeric bearings atop the pier and the other end supported by a set of temporary props. These 248t units are tandem lifted into position using one 300t and one 180t crane.

The remainder of the end spans are constructed using 3.5m long precast segments lifted into position and supported off a set of cantilevering beams so that a 600mm long in-situ stitch can be constructed between the new and previous units.

Construction proceeds in a balanced cantilever manner with segments installed each side of the pier and stressed on by cantilever tendons. The two boxes are stitched together longitudinally at the deck on centreline of bridge as cantilever construction proceeds. The cycle for installing a pair of segments and stitching them together takes one week. The method was chosen to be faster, to reduce labour and temporary works when compared to a falsework methodology often employed.

**Watercare Watermain**

The Alliance’s scope of work was extended after award to include construction of a 1.6m diameter concrete lined steel pipe watermain, supported between the two boxes on centreline of the bridge, and a precast concrete service walkway on both sides of the pipe. The 8m long 8 tonne pipe sections and 8m long 8 tonne walkway sections are installed as bridge segment construction proceeds by suspending the units beneath the bridge on a crane mounted frame and then lifting them into place with hoists acting through holes left in the deck.

Concrete walkway units, were originally ruled out due to the cost associated with marine plant over the duration of the project. Development of the installation methodology described above using the crane mounted frame, and utilisation of the staging, enabled the concrete units to be designed without the need for marine plant for installation. Concrete was preferred by the client to reduce long term maintenance.

**SUMMARY**

This project has been delivered by an alliance arrangement which provides significant opportunity for interaction between designers and constructors and the ability to discuss, clarify and modify if
appropriate the clients requirements. This has enabled significant innovation and led to material and programme savings without compromising the desired whole-of-life performance.

ACKNOWLEDGEMENTS

The authors wish to acknowledge NZTA and Watercare for their permission to publish this paper and the other members of the large team who have and are contributing to the New Mangere Bridge.
FIGURE 3 - TYPICAL GIRDER 1/2 SECTIONS