World Academy of Science, Engineering and Technology
International Journal of Civil and Environmental Engineering
Vol:18, No:8, 2024

Sustainable Upgrade of Existing Heritage Infrastructure: Strengthening and Rehabilitation of the LH Ford Bridge

Vince Scolaro, Lakshman Prasad, Ted Polley, Sanjivan Deshpande

Abstract**—**The LH Ford Bridge, built in the 1960's, comprises 28 spans, is 800 m long and crosses the Macquarie River at Dubbo, NSW. The main bridge spans comprise three spans with a 63 m centre span (25 m drop-in section) supported by halving joints from the main cantilevers and back spans of 28 m. The main bridge spans were built using complex construction staging (first of this type in NSW). They comprise twin precast boxes, in-situ reinforced concrete infills, and cantilevered outriggers stressed both longitudinally and transversely. Since construction, this bridge has undergone significantly increased design vehicle loads and showed signs of excessive shrinkage and creep leading to significant sagging of the centre span with evidence of previous failure and remediation of the halving joints. A comprehensive load rating assessment was undertaken taking account of the original complex construction staging. Deficiencies identified included, inadequate capacity of the halving joints, failure of the bearings at the halving joints, inadequate shear capacity of the girder webs and inadequate girder flexural capacity to carry B-Double design vehicles. A strengthening system comprising two new piers (under each of the halving joints), new bearings and installation of external prestressing to the soffit of both drop-in-span and back spans was adopted. A portion of dead load had to be transferred from the superstructure to the new piers via innovative soft/stiff bearing combinations to reduce new locked in stresses resulting from the new pier supports. Significant temporary works comprised a precast concrete shell beam forming the pile cap/pier structure, addition of temporary suspended scaffold (without overstressing the existing superstructure) and installation of jacking stays for new bearing top and bottom plates. This paper presents how this existing historic and socially important bridge was strengthened and updated to increase its design life without the need for replacement.

Keywords**—**Strengthening, creep, construction, box girder.

I.Introduction

THIS paper focuses on the capacity upgrade of the main three span structure from pier 21 to 24 over the river.

The three-span section comprises of pre-cast and in-situ concrete elements stressed together both longitudinally and transversely. This section of the bridge was showing signs of creep deformation despite being strengthened previously. In early 2012, RMS (now Transport for New South Wales) carried out a preliminary investigation/assessment of the existing structure which was independently peer reviewed. The study recommended two new supports be introduced for the drop in span.

The objective of the detailed design phase was to bring the capacity of the river section of the bridge (i.e., between Piers 21 & 24) to current standards for higher freight productivity vehicles while increasing the longevity of the bridge. Introduction of two new supports not only helped address some of the structural deficiencies identified in the existing bridge for Higher Mass Limit vehicles but also reduced future maintenance costs. The study also included the new locked-in effects from the new support arrangement, in addition to the development of solutions for strengthening the superstructure, for larger BD68 freight vehicles.

II.History of Existing Structure

The bridge over Macquarie River was constructed circa 1968-69. The bridge has 28 spans of varying lengths. Approach spans 1-21 and 25-28 consisted of 19.8 m spans with precast prestressed concrete I girders.

This paper covers the main river spans between piers 21-24. The three-span arrangement comprises of 28.0 m end spans and 62.8 m central span. The central span comprises of 19.32 m cantilever arms supporting a 24.12 m drop-in span This is as shown in Figs. 1 and 2.

The superstructure was erected using the balanced cantilever method. The two box girders were individually erected and stressed longitudinally. The central cell and cantilever sections were then erected, then the second stage longitudinal stressing was completed.

The box girders comprise of two precast box sections of varying depth, nominally 2.0 m long segments, and were longitudinally post-tensioned. The 75 mm wide joint between precast box elements are concreted with 41 MPa concrete. The central soffit in-situ slab was made monolithic with the two precast box girders.

The box girder cantilever slabs were precast and transversely post-tensioned with the central precast slab, forming the top deck of the bridge. The precast cantilever slab section comprises of 1.022 m wide inverted T segments with one transverse post-tensioning duct in each segment. The recess between the T-sections of the cantilever deck slab segments was filled with light-weight concrete as shown in Fig. 3.

Vince Scolaro is Director of Civil Structures Studio, Australia (e-mail: vince.scolaro@civilstructuresstudio.com).
Lakshman Prasad is Senior Bridge Engineer (Rehabilitation) with Transport for New South Wales.
Ted Polley is Technical Director with Beca.
Sanjivan Deshpande, Bridges and Structures Lead (South), is with Transport for New South Wales.

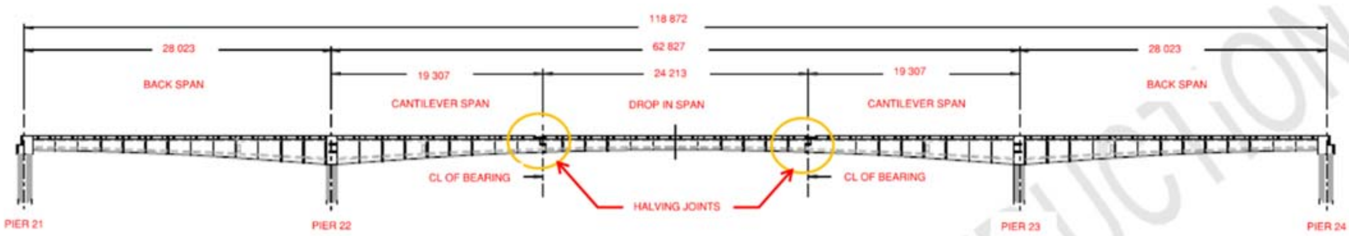

Fig. 1 Span arrangement between piers 21-24 including enlarged half-length of bridge section [6]

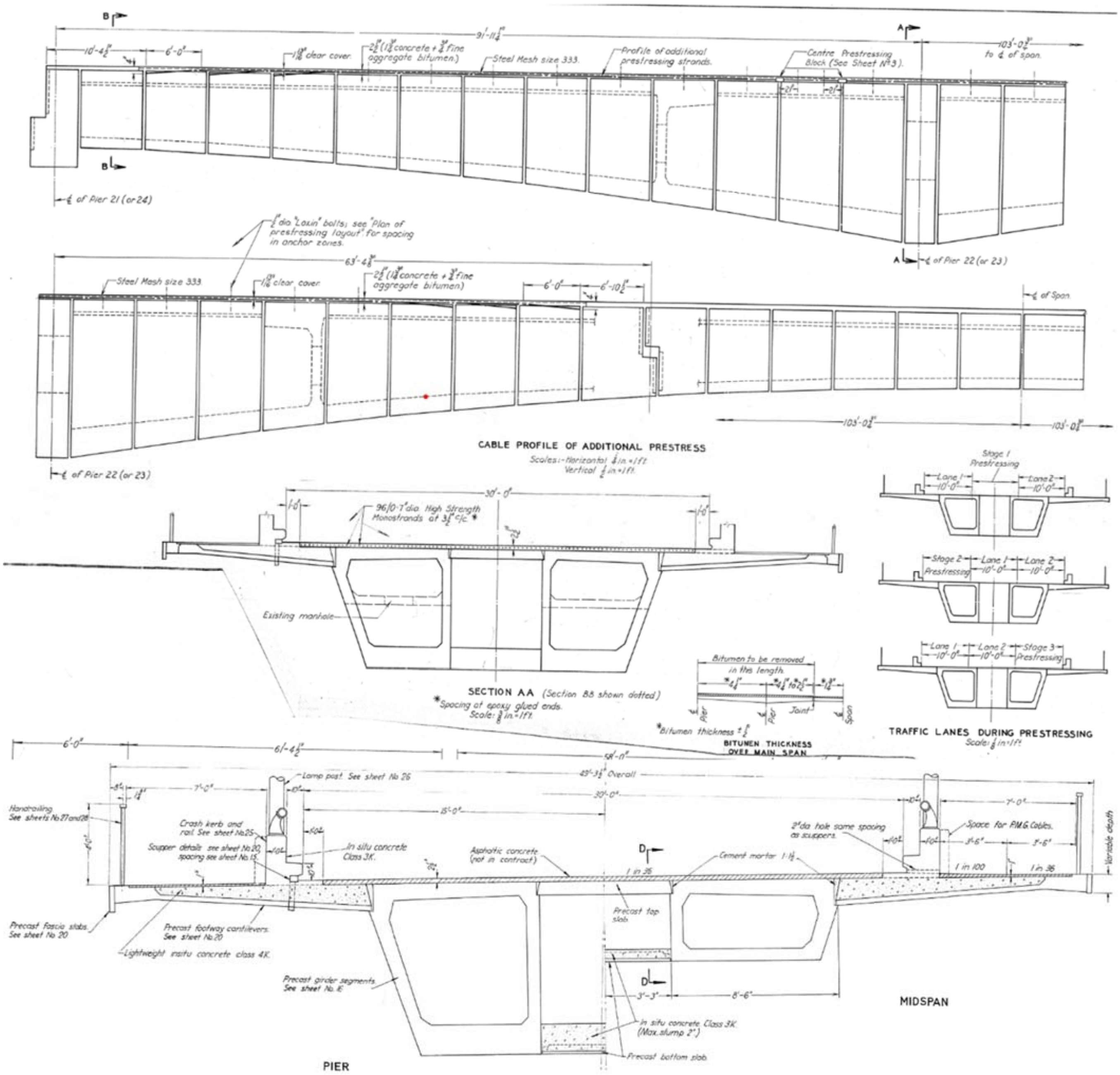

Fig. 2 Three cell varying depth box girder of spans 21-24 [6]

Each longitudinal tendon comprises of 11 strands of 0.5" (12.7 mm) dia. housed in a 60 mm duct. These tendons were grouted after stressing. There are 19 longitudinal tendons in each precast box units which were stressed in two stages. Transverse post-tensioning tendon comprises of 32 mm HT bar housed in a 41 mm dia. duct grouted after stressing. The three-cell box girder comprises of different concrete strengths as shown in Fig. 4.

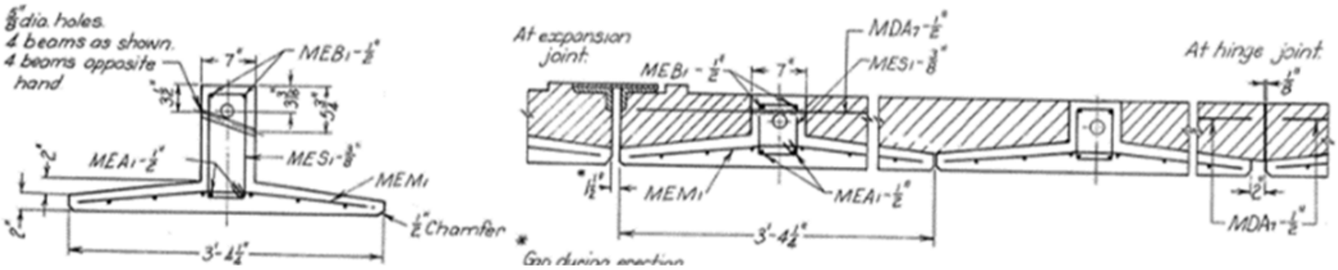

Fig. 3 Deck cantilever precast segments and in-situ concrete fill [6]

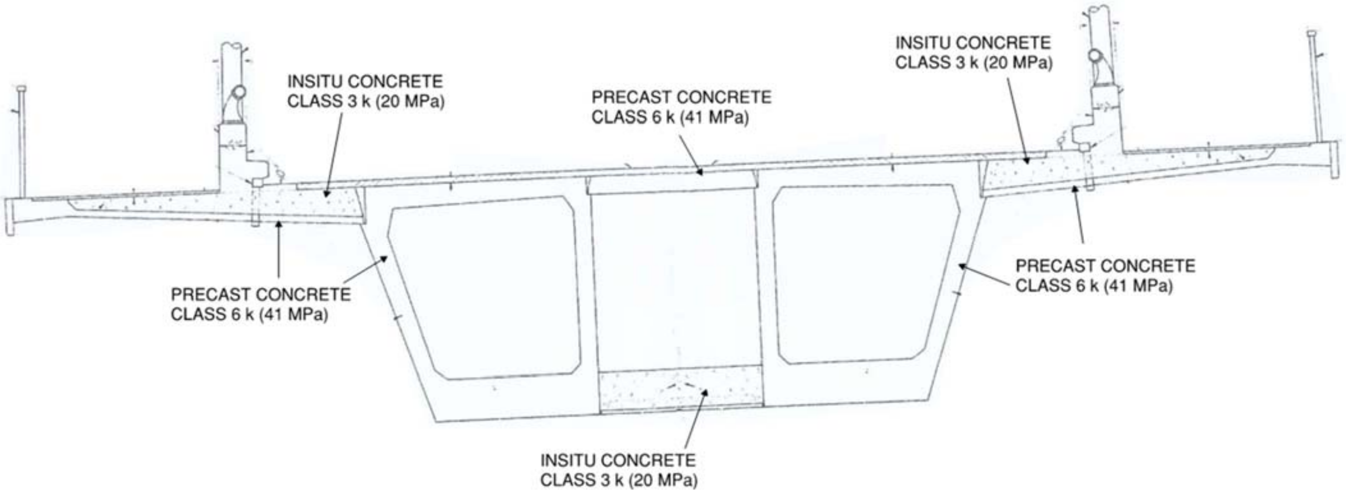

Fig. 4 Concrete strengths of the components of the three cells forming the box girder [6]

The balanced cantilever construction was carried out on a combination of scaffolding forms as shown in Fig. 5. The drop in span was constructed on temporary beams spanning between (and above) the cantilever span ends, then the drop in span was lowered into its final position.

The bridge was designed in accordance with the NAASRA Highway 1965 Bridge Design Specification [1]. The assessment was carried out using the AS5100:2004 [2] Bridge Design set of standards, including AS 5100.5:2015 Interim RMS Edition – Rev 2 [3]. Table I shows the estimation of long term effects of creep and shrinkage comparison specified in the NAASRA Highway 1965 Bridge Design Specification [1] and AS 5100.5:2015 [3]. It can be seen the original design assumed much lower losses, resulting in the primary reason for the creeping deflection at the cantilever tip.

TABLE I
COMPARISON BETWEEN LONG TERM LOSSES (DESIGN AND MODERN STANDARDS)

Long Term Effect	1965 NAASRA Highway Bridge Design Specification [1]	AS 5100.5:2015 (estimated avg.) [3]
Creep	450×10^{-6}	900×10^{-6}
Shrinkage	200×10^{-6}	650×10^{-6}
Relaxation	82 MPa	75 MPa

In 1973, the box girders were retrofitted with additional 94 – 0.6” (15.2 mm) and 0.7” (17.8 mm) strands. The strands were tensioned from one end. This was carried out due to considerable sag in the cantilevers after the construction. The strands were installed in slots at the top of the deck slab after removing the 64 mm thick asphalting surface. The extent of strands was located between the two traffic barriers. After stressing the strands were covered with a new 45 mm thick concrete layer and 19 mm thick surfacing layer consisting of fine aggregate bitumen. The arrangement of strands is shown in Fig. 6.

Figs. 7 and 8 graphs the sag of the cantilevers after the construction of the span between Piers 22 and 23. The recovery of sag after the additional prestressing installed can be observed in Fig. 8.

III. EXISTING STRUCTURE ASSESSMENT

The initial phase of the project was an assessment of the existing superstructure capacity to support large vehicles comprising of T44/L44 and BD68 vehicle loading and to determine whether strengthening was required.

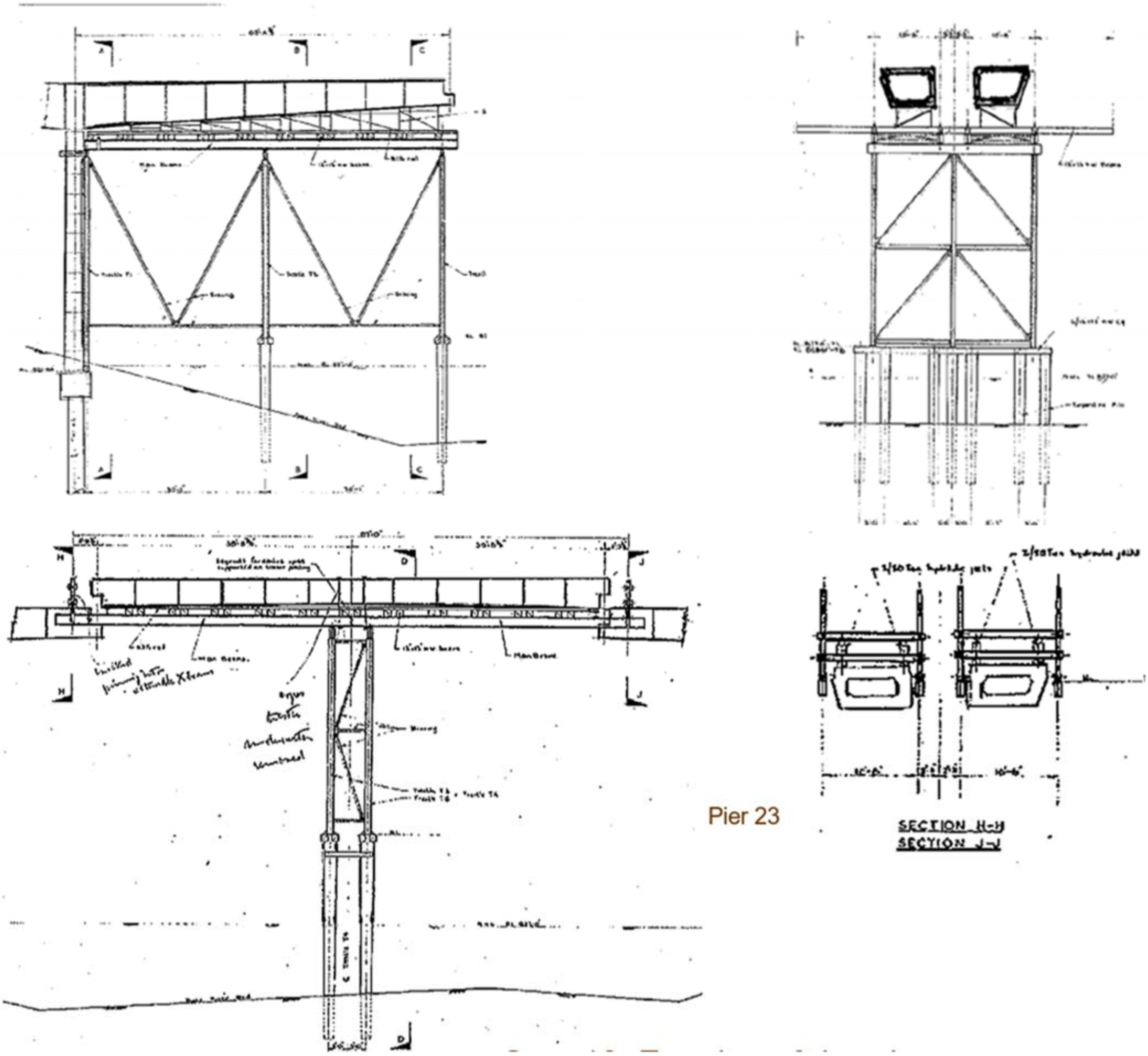

Fig. 5 Construction of spans between Piers 21-24 including Drop in Span Methodology [6]

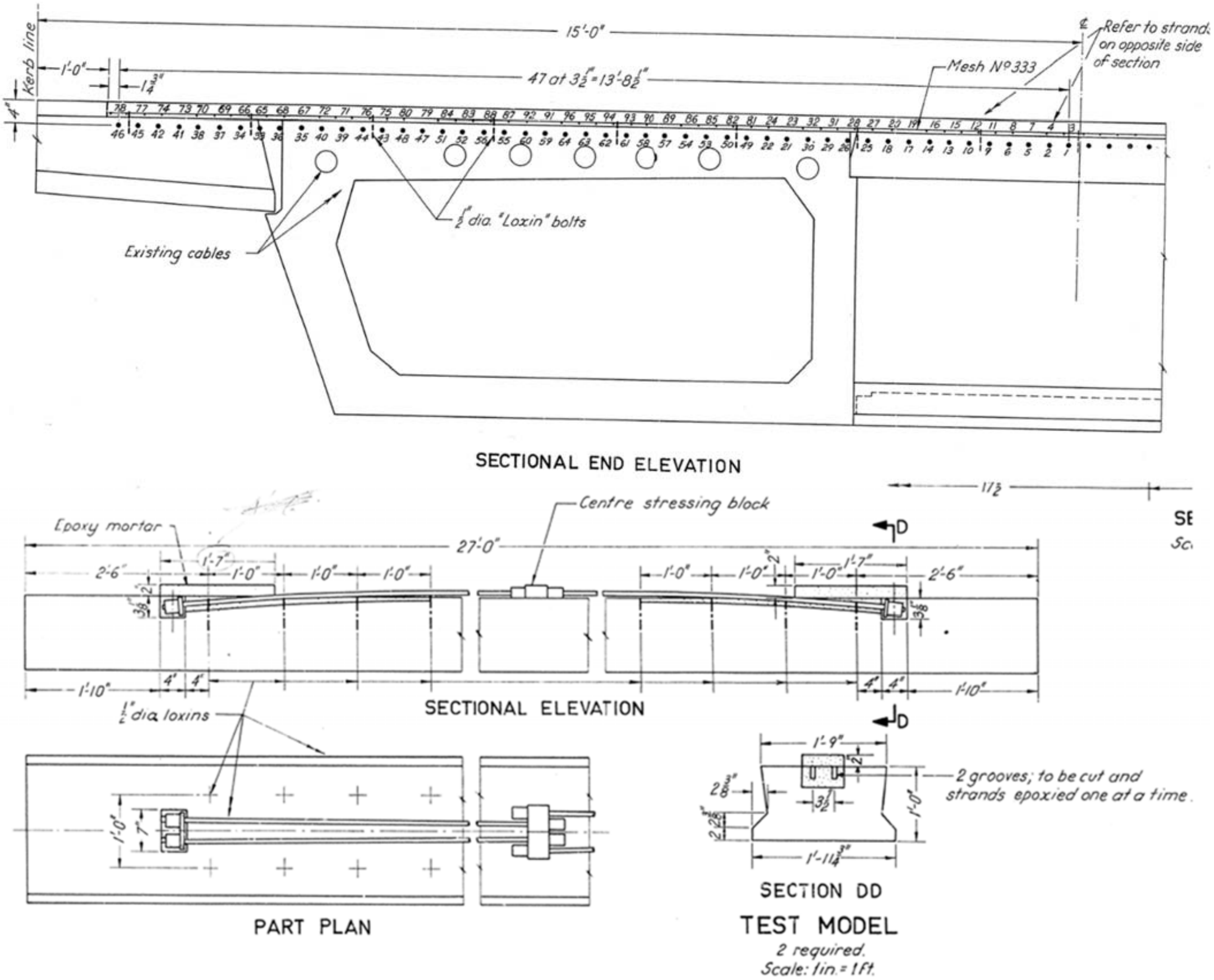

Fig. 6 Additional prestressing carried out in 1973 [7]

The bridge was assessed using a combination of:

- A single spine beam element model (using SAP software) to assess the effects of Dead Load, Superimposed Dead Load, Temperature, and other effects,
- A shear-flexible grillage model as outlined in Bridge Deck Behaviour by Hambly [4] to assess Live Load (Vehicle) effects.

The existing bridge piers were modelled assuming fixity at the base of the piers and connected to the superstructure via pot bearings.

The shear flexibility of the grillage model reproduces the distortion behaviour of the box girder cells and is particularly appropriate for wide multicellular decks. The model directly derives the distribution of vertical shear and torsional shear components in the girder webs and distribution of flexural stresses across the bridge deck.

Section properties were transformed to allow for varying concrete strengths used in the various components comprising a bridge segment and shear lag was allowed for in the bridge cantilevers in accordance with the provisions of AS5100.5-2015 [3].

Torsion stresses were turned into shears and designed using the shear provision in AS5100.5-2015 [3] as the torsion provisions were considered too conservative for an existing bridge assessment.

In accordance with the design and assessment standards the capacity of the bridge segments for flexure and shear was assessed to comply with AS5100.5-2015 [4]. The following demands were assessed:

- Serviceability Limit State (SLS) Flexural Stresses including prestressing losses calculated in accordance with the staged construction sequence provided by TfNSW.
- Ultimate Limit State (ULS) Flexure (as Live Load flexural stresses vary across the deck, the worst case "web section" were considered at each bridge segment)
- ULS Web Shear and ULS Combined Web Shear and Transverse Bending (including Box Distortional effects)

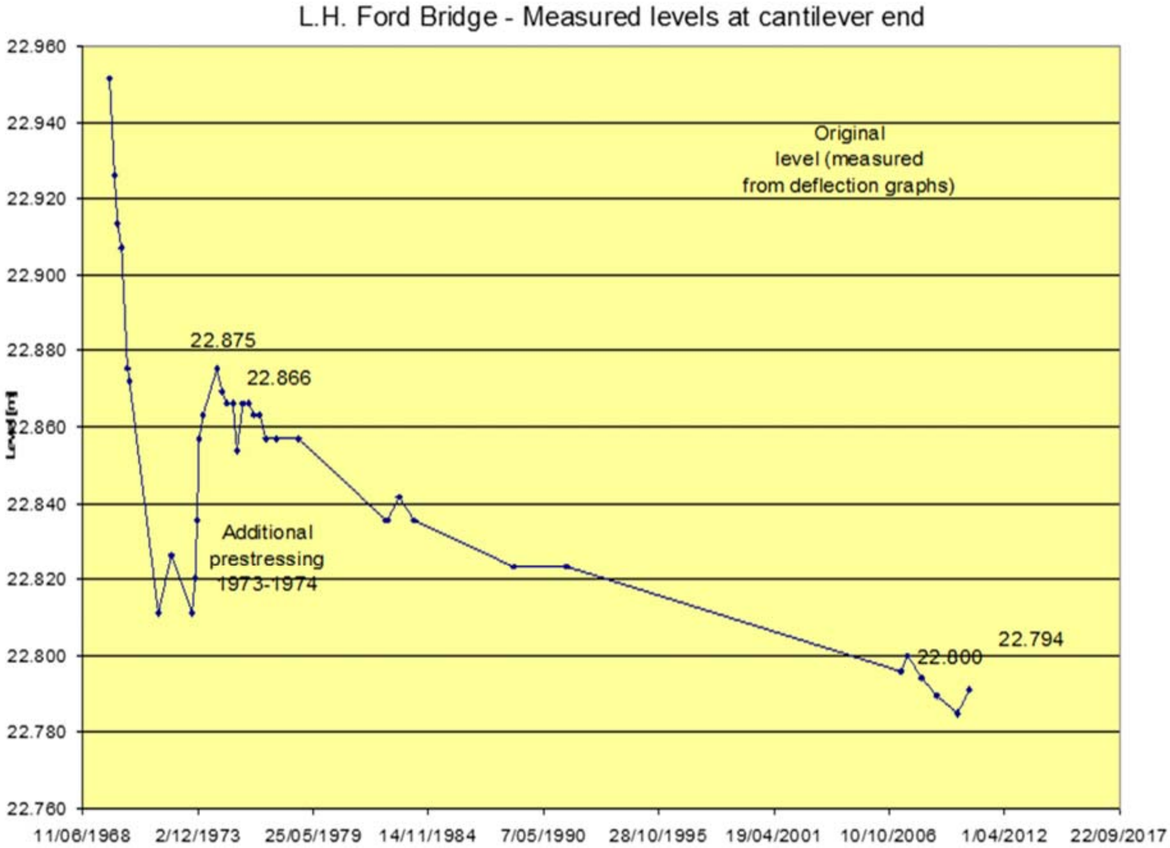

Fig. 7 Timeline of cantilever sag after construction

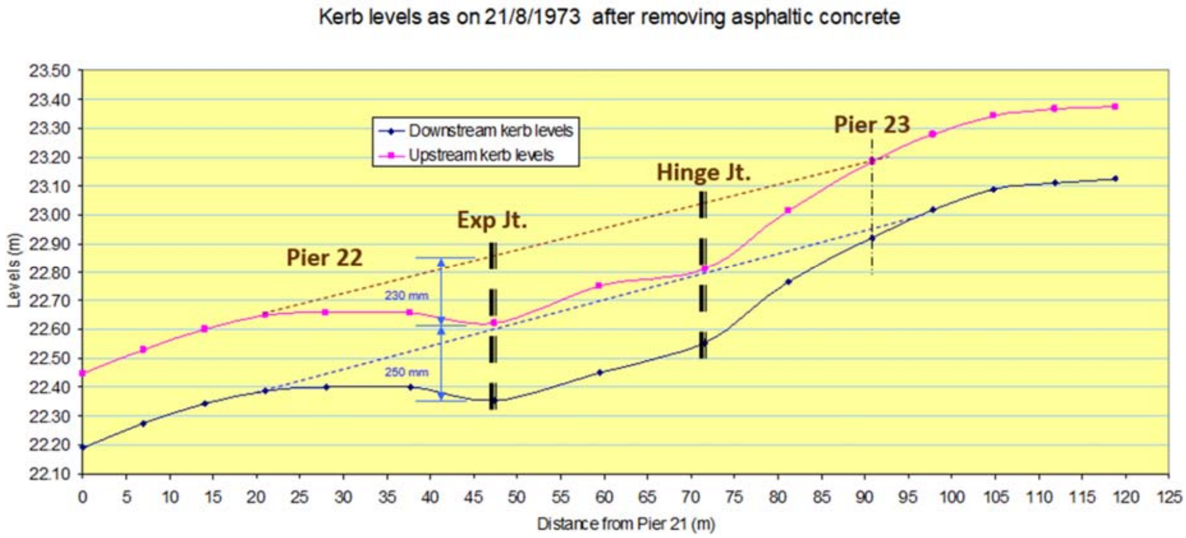

Fig. 8 Pre-strengthening profile of bridge

The halving joint detail was assessed and found to be overstressed under ultimate dead and live load demands. Strengthening of a halving joint was considered difficult to achieve due to:

- High concentrations of load from post tensioned tendon anchorages combined with high vertical shears from the drop in span and,
- Limited construction access

The assessment concluded that strengthening of the half joints and deck drop-in-span was required.

Numerous concept options were considered and submitted for community consultation that involved urban design considerations as well as heritage considerations.

From a structural perspective, two piers, one located directly under each halving joint was considered the best solution, particularly for the halving joint. The additional benefit of the piers was that the cantilever span could be jacked upwards to remove a proportion of the historical vertical creep deflection.

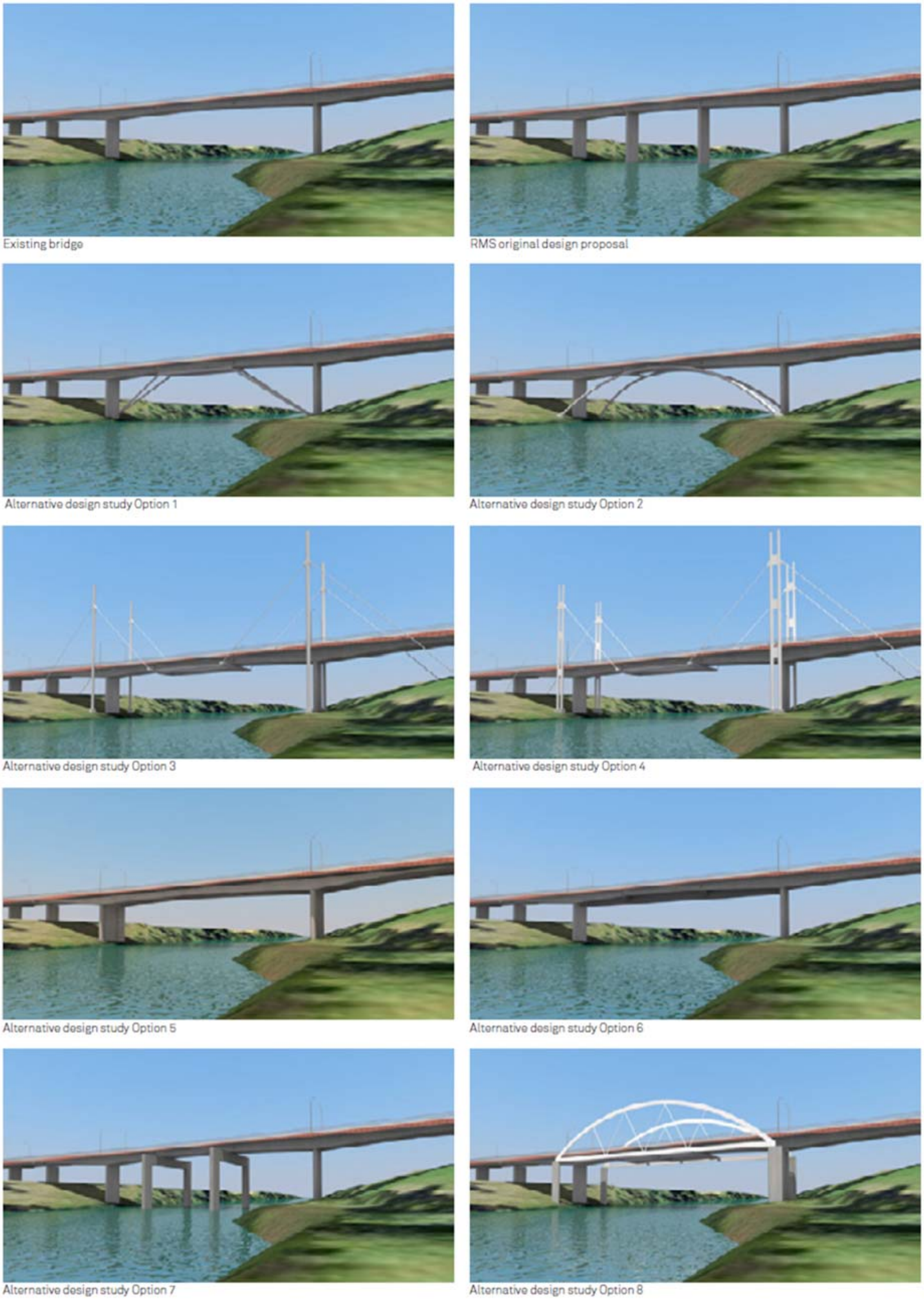

Fig. 9 Concept Options Considered

IV. Stakeholder Consultation

Since the LH Ford bridge is a significant Bridge located at the entrance to the major city of Dubbo, the project naturally attracted a lot of community interest and involvement. The city is located at the intersection of Newell, Mitchell and Golden Highways and is a major road and rail freight hub to other parts of New South Wales. Several concept options, as shown in Fig. 9, were developed with varying visual impacts. Following the outcome of community consultation and keeping the safety of the community in mind the option of two piers was preferred which is simple, elegant, and cost effective that blended well with the profile and configuration of the existing bridge.

V. Detailed Design

The detailed design stage was carried out considering T44/ L44 and BD68 vehicle loading including vertical support now provided by the new piers. Key assumptions for the detailed design were

- The structure was assumed to be constructed as per the drawings and was in good condition.
- The shear keys between precast elements and construction joints between in-situ and precast elements were considered adequately roughened to allow shear transfer across the joint. The mortared joints were also assumed to be in good condition, allowing full load transfer through the joint. This assumption was confirmed on site by a visual examination of the mortar.
- The structure was built as per the construction sequence provided by TfNSW based on historical reports.
- Concrete barrier and concrete overlay covering the strengthening strands were not composite with the box segments.
- All elements of the existing stressing systems were assumed to be in good condition and all cantilever tendons were assumed to be grouted. All external strands were assumed to be un-bonded.
- Pedestrian loading was not to be considered in combination with roadway traffic loading. This was a key assumption as this reduced the shear and torsion demands on the girder and hence reduced the extent of strengthening.

In addition, an assessment of the long-term concrete compressive strength of the bridge was carried out by TfNSW. This comprised the taking of cores from the soffit of the precast box sections of bridge for testing and additionally a series of Schmidt Hammer tests. The long-term compressive strength of the bridge concrete was assessed at 46 MPa (compared to 41 MPa design strength) and was used for updated member capacities.

Load Transfer Design Philosophy

The design used an innovative combination of softer bearings on the cantilever side and stiff bearings on the drop in span side at each new pier. The softer bearing was used for two reasons. The first reason was so live load on the drop in span will go into the new pier rather than travel across the existing halving joint (known to be deficient) and then into the new pier since load travels through the most direct and stiffest load path.

The second reason is to create a self-levelling system. The existing cantilever tip has been creeping downwards due to the amount of cantilever prestress combined with the dead load of the cantilever and drop in span. Should the cantilever continue to creep downwards after the new pier was installed, the softer bearing would compress and allow a small vertical movement to occur (1-2 mm). The adjacent stiff bearing would then carry the entire drop in span dead load since it does not compress. The cantilever, being relieved of this load, would start to deflect upwards, reengaging the drop in span and picking back up the drop in span dead load.

The stiff bearings were preloaded by jacking up the cantilever (approximately 25% of the drop in span dead load) during bearing installation. The 25% jacking case was found to minimise the flexure stresses on the existing backspans while reducing the drop in span dead load reaction transferring through the deficient halving joint. Provision for future jacking has been provided. It is anticipated that the jacking loads may creep away over time, resulting in reduced/negligible preload in the stiff bearings.

Substructure

For the detailed design of the new piers, the structure was modelled using SpaceGASS software in both the longitudinal and transverse directions.

Longitudinal Model

A two-dimensional beam model was used to determine actions in the longitudinal direction.

The superstructure was represented by a single line member with section properties calculated for each of the precast segment lengths. The existing piers were included in the model as single line members with corresponding section properties. The existing pile foundations were assumed to have a point of fixity at a depth of 3 x pile diameters and modelled accordingly.

The new piers and piles were modelled as two-dimensional line members. The following two scenarios were considered for the fixity of the new piles:

- Pile fixity of 2.5 x pile diameters below the scour level (scour level 3 m below existing riverbed).
- Pile fixity of 3 x pile diameters below the scour level (scour level 5 m below existing riverbed). This corresponds with the existing rock level and assumes that all material above rock is scoured.

The articulation of the bridge was modelled as shown in Table II.

TABLE II
BRIDGE ARTICULATION

Pier	Location	Type of Bearing	No. of bearings
Existing Piers 21-24	Centred over pier	Existing Steel Hinge (fixed in translation)	4 per pier
New Pier: Dubbo end	Cantilever side	Elastomeric	4
	Drop-in span side	Free float pot bearings	4
New Pier: Narromine end	Cantilever side	Elastomeric	4
	Drop-in span side	Free float pot bearings	4

Note: transverse and longitudinal restraints at the two new piers are resisted by new steel shear keys. This was to avoid large hold down bolts (for fixed/guided bearings to be used) being installed into the existing superstructure webs at the location where the longitudinal post tensioning anchors occur.

Vertical load from dead load and superimposed dead load were included to provide design actions on the new piers and foundations in the longitudinal direction.

Three live load scenarios were considered for each design vehicle as listed below:

- Scenario #1: Maximum compression in new pier
- Scenario #2: Maximum bending in new pier (toward drop-in span)
- Scenario #3: Maximum bending in new pier (toward balanced cantilever)

Longitudinal forces on the drop in span (braking and thermal) were transferred to the new pier at the Narromine end via a shear key. To avoid excessive loads on the new pier, braking forces at this pier were transferred back to the cantilever span superstructure via a second shear key.

Transverse Model

A three-dimensional model of the new pier and foundations was used to determine design actions in the transverse direction.

As per the longitudinal model, two scenarios were considered for the fixity of the new piles:

- Pile fixity of 2.5 x D below the scour level (scour level 3 m below existing riverbed).
- Pile fixity of 3 x D below the scour level (scour level 5 m below existing riverbed). This corresponds with the existing rock level and assumes that all material above rock is scoured.

Jacking loads comprising 30% of the vertical loads due to superstructure dead and superimposed dead loads of the drop in span were assumed to be distributed between the four new bearings. Hence these loads were applied as an axial compression to the transverse substructure model.

The critical transverse live load scenario was applied to the model as a compression and equivalent moment to account for the eccentric placement of design lanes.

Thermal loads and the 500 kN minimum restraint load were also considered as part of the transverse model.

Transverse loads on the superstructure were transferred to the substructure via a shear key at both new piers.

Superstructure

SLS checks were made at every section of the bridge in accordance with the construction sequence provided by RMS. The construction sequence was a staged process with eight tendons in each precast box girder section being stressed longitudinally before the precast cantilever sections and central slabs were introduced. The remaining 11 tendons (per box) were then stressed once the full superstructure cross section was formed. This meant that the precast box sections initially had a higher level of prestress than the deck cantilever and the central slabs during construction. Over time the prestress will have tended to redistribute across all the elements due to creep. It should be noted that there is no staged stressing assumed for the drop in span.

Each assessed section considered two bounds. The lower bound assessment assumed no stage one prestress (first eight longitudinal tendons) on the basis that the precast concrete box section stress had crept into the cantilever and central slab sections. The upper bound assessment assumed the first stage prestress was applied to all sections. It can be expected that the actual stress state is somewhere between. The drop-in span was constructed entirely on false work, then stressed, and then dropped into place and so there is no requirement to consider an upper bound.

The criterion for these checks is zero tension at the construction joints for the precast sections as there is no reinforcing crossing these sections.

The initial assessment stage found that jacking the bridge by 25% produced the best results for modifying the stresses in the back span. The analysis was further bounded by two jacking scenarios. The first is a jacking load of 30% dead load and the second scenario is if the entire jacking load has crept out and is zero.

Shear demands were derived from the single spine beam model for dead Load, superimposed dead load (SDL), temperature and jacking effects. The shear demands taken from this model were based on the global shear being divided by the four webs. In addition, there was a small torsional component of dead load caused by the curvature of the bridge. Since this was small in comparison with the torsional component of eccentric live load this effect was assumed to be distributed as a moment couple between the outer webs. This assumed that for dead load torsion, St Venant's torsion theory of shear flow around the perimeter of the section would apply.

Shear and torsion demands due to live load were derived from the shear flexible grillage model. This model provided the distribution of torsion and shear demands across the bridge superstructure. Since there was no regular cross bracing or continuous diaphragms between supports to prevent the box girder cells changing shape by distortion, it was necessary to take the additional flexibility into account. A byproduct of this flexibility was an uneven distribution of shear stress, particularly for eccentric loading.

The capacity of the box girder webs was assessed using the approach summarized in Chapter 5 of Prestressed Concrete Bridges by Christian Menn [5]. The major difference was that Menn assumes the vertical shear is carried by the stirrups only with no concrete (Vc) component resisting the demand. This was considered too conservative for an existing structure and would have meant that the girder webs failed in pure shear before adding local bending effects. The assessed capacity was therefore calculated after deducting the concrete resisting contribution (Vc) from the demand (Vc was determined from the global shear capacity check and hence consistent with both global and local checks).

In accordance with AS5100.5-2015 [3], the vertical component of prestressing and vertical component of compression stress in the inclined box girder bottom flange was considered to contribute towards vertical shear capacity in the webs where applicable. An increased calculated shear strength capacity was adopted utilising the long-term concrete compressive strength, which was based on a statistical analysis of concrete cores and Schmidt hammer test results taken along the bridge deck.

Deck Slab

The deck slab comprised of one metre nominal width precast slabs placed between the precast box sections. These were stressed to the box sections at the same time as the precast cantilever units.

A simple two-dimension three cell box model was created to determine the bending moments throughout the cross section. Each web was restrained vertically in the model. The distortional box moments were generated from the shear flexible grillage model and added to the local model results. Two sections were checked, adjacent to the pier and at midspan of the drop in span. This gave the two limits of transverse moment which was distributed into the webs and flanges from the deck moments. Wheel loads were assumed to be carried by 1 m effective width. The sagging moments at midspan of the precast top slab sections were treated as partial prestress members and the stresses were found to be less than 150 MPa and therefore considered acceptable.

Superstructure Strengthening Solution

The drop in span was found to be overstressed in flexure at ULS and SLS. The solution was to install four tendons on the outside of the box girder to increase the flexural capacity. Due to limited access and headroom, installing external tendons on the inside of the box girder would have been extremely difficult.

Steel anchorage pods were installed on the outside, stressed onto the webs near the bottom of the existing box girder. Prestressing strand tendons were be used to enhance the flexural strength. The tendons had a double corrosion protection system to provide 100-year design life. The tendons were detailed so they could be replaceable if required.

Due to the risk of the backspan becoming overstressed if the bridge was jacked up too high, four external tendons were installed on the backspan. The backspan tendons were installed before the new piers were constructed to avoid secondary prestressing effects being locked into the structure.

New steel shear keys were required to provide lateral restraint between superstructure and the new piers. The shear key comprised of a Square Hollow Section (SHS) which was bolted to the pier and the shear guide restraints stressed to the superstructure soffit.

As noted previously, the existing halving joint stresses were reduced by the implementation of the stiff and soft bearings at the new piers. The drop in span dead load reaction crossing the halving joint was reduced by approximately 30% and all live loading on the drop in span was now transferred through the bearing into the pier.

Refer to Fig. 10 for final strengthening solution.

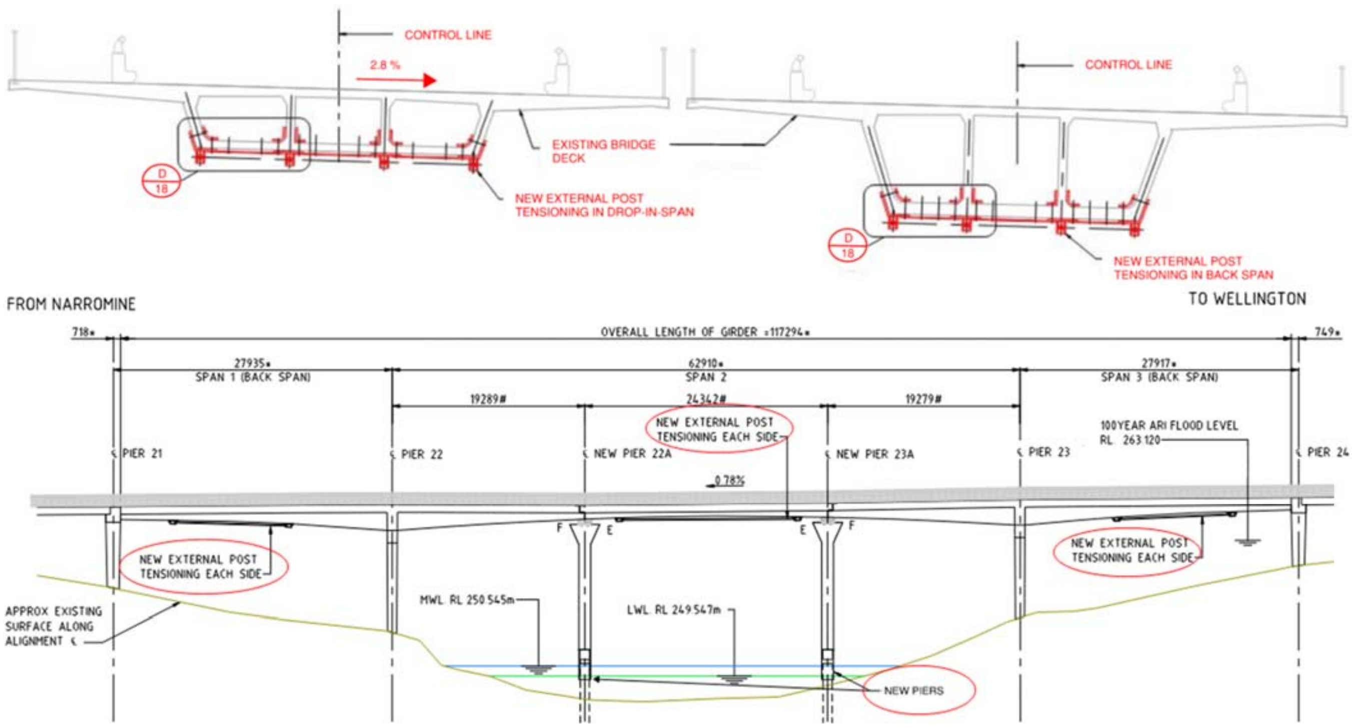

Fig. 10 Final Strengthening Solution [8]

VI. Construction

The works were carried out by Freyssinet Pty Limited in 2019 and 2020.

The construction sequencing of the strengthening works was an important input into the design, particularly timing around the external post tensioning and the jacking. The summary of the sequencing was as follows:

- Install and stress backspan external post tensioning

- Construct new piers and position bearings (refer to Fig. 11)
- Install temporary works for new pier position adjustment during jacking
- Close bridge
- Jack bridge off new piers and lock in load on jacks
- Open bridge
- Grout new bearings and cure
- Transfer load from jacks onto bearings
- Install and stress drop in span external post tensioning

Fig. 11 Construction of the new piers

Temporary Works

Significant temporary works were required for the construction of the strengthening works. These works comprised:

- Assessing the existing bridge for temporary hanging scaffolding (refer Fig. 11). Numerous iterations with the contractor were required to optimise the hanging support points so as not to overload the unstrengthened bridge. It was determined through a risk assessment that a live load of T-44 equivalent vehicle together with the self-weight of the scaffolding plus construction live load be adopted for this stage of work.
- Provision of real time modifications to the stressing anchorage brackets and bearing attachment plates (and associated steelwork) because existing reinforcing bars within the precast box sections were not to be cut. As such, all anchorage bolts were located through trial drilling to avoid the existing reinforcement (refer Fig. 13).
- Jacking the bridge and supporting the load on temporary jacks until the bearings were grouted resulted in some large eccentric vertical loads applied to the new piers causing the new pier to deflect longitudinally. To counter this effect, as well as bridge superstructure thermal movements, temporary diagonal restraints were deigned connected between the top of the new pier and the bottom of the adjacent existing pier as shown in Fig. 12. These restraints comprised of two tendons per pier, and they were tensioned to produce a lateral deflection opposing the expected jacking deflection. This was required so the bearing top and bottom plates could be installed. As the jack load was transferred to the bearings, the tension load in the temporary restraints was released to keep the new piers in their design position.

Fig. 12 Installation of diagonal restraints

Permanent Works Installation

Installing strengthening works to an existing structure required working through the constraints caused from:

- Limited design information including difficult to read or missing construction drawings
- Additional reinforcing or reinforcing constructed which differs from the available drawings
- As built dimensions that were different to that shown on the available drawings
- Limited access

The design considered these issues when detailing the anchorage pods for the external post tensioning and installing the shear keys. The anchor pods were stressed onto the box girder soffit adjacent to the webs using an innovative stressing technique. Using conventional post tensioning stressing systems would not work due to the short free length of the stressed bars and the limited access for jacking equipment. The solution was to use grade 8.8 threaded bars which were torqued using load indicating washers to produce accurate clamping forces which fixed the new anchorage pods/shear keys in position. These are shown in Fig. 13.

Fig. 13 Installation of anchor pods and shear keys

The anchorage pod installation was difficult due to the congestion of existing reinforcing in the webs and soffit. The design was detailed to be modified on site as required to reflect the As Built position of the bolt locations.

A jacking trial was carried out by the Contractor due to the potential risk of the bridge performing differently to the predicted forces and displacements from the analysis. The jacking load distribution across the four webs was similar to the analysis, but the new pier displacement was stiffer than expected.

VII. Conclusion

This paper provides a simple, yet robust solution to an age-old problem for bridges constructed using the non-continuous balanced cantilever method which are not dead load balanced and have a history of increasing cantilever creep deflections. The unique self-levelling system comprising of soft and rigid bearings will continually adjust the cantilever tip from further deflection. At the same time, this system significantly reduced the demands on the overstressed halving joint.

Acknowledgments

The permission granted by Transport for New South Wales (previously RMS) to publish the information contained in this paper and the support of Beca and Civil Structures Studio to present it at this conference is gratefully acknowledged.

This paper represents the opinions of the authors and is the product of professional work. It does not represent the position or opinions of Transport for NSW.

References

[1] NAASRA Highway 1965 Bridge Design Specification.
[2] AS 5100:2004 Bridge Design Standard (Parts 1-5 and 7)
[3] AS 5100.5:2015 Concrete Interim RMS Edition – Rev 2
[4] Bridge Deck Behaviour by EC Hambly – 2nd Edition 1991
[5] Menn, C et al. (1990) *Prestressed Concrete Bridges*, Birkhäuser Verlag, Basel, Germany.
[6] LH Ford Bridge design drawings dated 5.10.1967
[7] LH Ford Bridge design drawings dated 31.5.1973
[8] LH Ford Bridge design drawings dated 18.2.2015