AN AUSTRALIAN FIRST SOLUTION TO DELIVER KEY INFRASTRUCTURE IN CHALLENGING RAIL ENVIRONMENTS

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Abstract:

The Level Crossing Removal Project: Caulfield to Dandenong constitutes the removal of nine of Melbourne's most congested level crossings along the Caulfield to Dandenong rail corridor. Grade separation of the first four level crossings of this corridor, between Caulfield and Hughesdale was achieved by elevating the existing at grade rail infrastructure onto a pair of newly constructed 3.1km long viaducts. This section of the rail corridor, within which Melbourne's busiest rail line operates, is heavily constrained by limited access and narrow working areas and thus necessitated the development of an innovative solution for project delivery. Precast segmental construction was adopted to minimise the disruption of existing passenger and freight rail services throughout the construction period, and involved the implementation of a modified full span installation technique. The use of this method, implemented with the aid of a specialised Straddle Carrier and Launching Beam system, makes it the first project of its type to adopt such an erection scheme in Australia. The segment casting, delivery, assembly and erection engineering was a substantial undertaking and involved the establishment of a purpose built casting yard in Pakenham, an assembly yard at Murrumbeena Station and the implementation of several items of specialised plant and temporary works. All 87 spans of each of the elevated viaducts were built immediately adjacent to the existing rail lines with little all or no impact on continuing rail operations, or major disruption to the local road network. New rail infrastructure attached to the viaducts includes overhead wiring support gantries, rail signal structures, train stops, direct fixed track and a combined services route. In addition, the viaducts support three new elevated stations with island platforms shrouded by an architectural canopy at Carnegie, Murrumbeena and Hughesdale.

Keywords: Rail Viaduct, Precast Segmental Box Girder, Straddle Carrier, Span by Span erection.

i. Introduction

The Level Crossing Removal Project: Caulfield to Dandenong was established to remove nine of Melbourne's most congested level crossings along the Caulfield to Dandenong corridor and completely rebuild five stations on Melbourne's busiest rail line by late 2018. The project involves the lengthening of existing platforms, in addition to new signalling and power systems to support new longer High Capacity Metro Trains affording a significant increase in capacity.

The Level Crossing Removal Project (LXRP) identified the following Project Objectives:

- Maintain an acceptable level of service for road and rail users during delivery
- Improve the reliability and efficiency of the transport network to improve productivity
- Align with community and stakeholder expectations
- Provide early engagement with stakeholders to take them on the journey
- Promote appropriate land utilisation around rail corridors to facilitate value capture development rights opportunities

- Provide better connected, more vibrant activity centres and improved urban amenity for all users and create safer communities

The project was delivered by the Caulfield to Dandenong Alliance (CTD), consisting of Alliance partners Lendlease, CPB, Aurecon, WSP Parsons Brinckerhoff, MTM and LXRP.

The major elements of the project included:

- Three sections of elevated structure,
- Removal of the level crossing infrastructure and refinement of the existing road alignments,
- Five reconstructed stations at Carnegie, Murrumbeena, Hughesdale, Clayton and Noble Park,
- Extended platforms at other stations along the line,
- New rail systems and signalling along the greater alignment stretching from the City-Loop out to Cranbourne and Pakenham,
- A linear park beneath the elevated structures and a shared user path running along the full length of the alignment.

Fig. 1 Extent of Level Crossing Removal Project Pakenham Cranbourne Corridor

The project was split into three main areas: Area 1 between Grange Road and Poath Road; Area 2 between Clayton Road and Centre Road; and Area 3 between Corrigan and Chandler.

This paper focuses on the main challenges of the heavily constrained 3.1km elevated section of Area 1 for which VSL was contracted in a Sub-Alliance. The new elevated structures in this area carry rail over the four existing level crossings between Caulfield and Hughesdale.

ii. The Search for a Solution

a. Grade Separation by lowering rail under road

Level crossings have historically been removed in Melbourne by lowering the tracks into an open trench and utilising bridge structures to carry road traffic over the trench at discrete locations.

However, a rail-under-road solution was not pursued for the Caulfield to Dandenong section for the following reasons:

- Large number of existing underground services within the rail corridor including major gas transmission lines, sewer and water mains. These would be time consuming and expensive to relocate with an open trench solution in a restricted working corridor.
- Discrete rail-under-road solutions at each of the level crossings would cause an undesirable 'roller coaster' effect in rail geometry and lead to inefficient train operations.
- The longitudinal gradient of the tracks must be kept relatively shallow, which for individual level crossings, results in trench structures exceeding 500m in length. As the level crossings in the Caulfield to Dandenong section are within close proximity to one another, the resulting trench would need to be continuous with the need for significant soil retention systems.
- The continuous length of the trench would essentially divide communities
- Natural overland flood paths would be cut-off
- There is a high water table along the corridor. An open trench solution leads to temporary and permanent changes to groundwater conditions which may result in undesired settlements of the neighbouring properties, predominately residential. Construction in this environment is challenging and would require continuous de-watering and treatment of contaminated groundwater. The trench structure must be designed for buoyancy and be suitably detailed to deal with long term drainage.
- Trench construction would require a significant number of prolonged rail closures to complete the works.
- The necessary excavation works would generate a large number of truck movements, placing further strain on the congested local road network and significantly disrupt local residents and businesses.

b. Initial Planning for rail over road

Super-T Girders Options

In Australia the most common bridge superstructure form consists of precast prestressed Super-T girders with an insitu deck slab. This system is well suited for short span bridges where site access for cranes is unrestricted.

This solution was adopted for the removal of all level crossings in Areas 2 and 3 where the rail corridor is wide, with good access and few obstructions to limit works. In this area, the delivery of materials and plant to site, and the use of large scale cranes for beam installation could easily be accommodated.

In Area 1, the Super-T construction method could not be employed due to the following reasons:

- Fully operational live railway within the construction corridor and restrictive clearances to residential properties bordering the project boundaries.

- Girder erection via conventional crane erection was not possible. Rail occupations would be required to set-up cranes, with the need to temporarily demobilise some rail infrastructure at certain locations.
- Casting of the in-situ deck slab of the bridge significantly increases the amount of on-site work adjacent to live tracks and adjoining residential properties.
- The maximum span length was limited to 30m for the heavy railway loading, and thus a high number of substructure sets are required to be constructed under difficult conditions.

An alternative erection scheme was also considered whereby Super-T beams would be delivered along completed spans and launched with an overhead gantry crane. However, the deck slab cast on top of the Super-T beams must achieve sufficient strength to allow span delivery to the working front. This would have resulted in slower span erection rates and was deemed not suitable in Area 1 as the programme for the construction of the viaducts was on the critical path of the project.
These programme constraints and the difficulties of in-situ construction above and adjacent to an active rail corridor led to the consideration of a pre-assembled full span Super-T erection method. Studies of this concluded that delivery of long precast elements into the site was too restrictive and the limited space on site and programme constraints rendered a full span Super-T option with a deck cast at ground level not feasible.

Other Bridge Forms

Other bridge construction technologies such as balanced cantilever erection and Incremental Launching Method (ILM) were quickly discarded due to:

- the number of tasks required to be performed at close proximity to the railway lines,
- the programme constraints resulting from the curing time of insitu concrete works

Preliminary Segmental Span by Span Solution

The first fully precast option that was developed as the tender solution for the Area 1 viaducts involved a series of precast segmental, post tensioned box girder spans, placed using the segmental span by span technique by three number single truss overhead gantries.

These gantries were programmed to work in different areas on a 24 hour per day, six days per week programme and required segments to be delivered along the working corridor to the point of installation. The development of a precast segmental solution allowed concentrated work fronts and provided early release of spans for follow up works. This form alleviated the constraints of insitu concrete placement adjacent to rail, and the programme implications of curing. The design of the substructure was driven by the need to provide sufficient horizontal and vertical clearance for construction plant and material delivery between the piers of the new works and the project boundaries, and between the piers and the fully operating rail. Permanent stressing of the external tendons was planned to occur with the segments suspended from the gantries, after which the completed spans were lowered onto temporary supports, before being finally lowered onto the grouted permanent pot bearings.
The proposed form of the overhead gantries was based on similar recent VSL experience for an elevated rail structure in Singapore. The use of a single truss gantry minimised the width of this type of equipment, and allowed segments to be unloaded, raised and suspended in place without any encroachment of plant or segments over the project boundaries or within the rail clearance envelope. However, this gantry method required strict controls for working at height operations and necessitated follow up works for the installation of the superstructure parapets. This involved insitu works and additional heavy lifting close to the operating trains once the spans had been completed.

Fig. 2 Tuas MRT overhead gantry (VSL) erecting a Segmental Span by Span superstructure

Full Span Segmental Span by Span

At project award, a thorough review of the original tender solution, including reassessment of conventional construction methods was initiated, with consideration of the following key constraints and objectives:

- Site is heavily constrained with restricted access at designated points of the corridor. Erection of heavy precast girders is impracticable with standard cranes.
- Very tight construction programme.
- A reduction of the occupational, health and safety hazards to workforce but also to nearby rail commuters and local residents – a key driver to minimise insitu works on site.
- Reduction in the number of piers by increasing span lengths
- Erection of most elements without impacting on rail or road traffic, or encroaching property boundaries.
- An aesthetically pleasing form for the finished product.
- Reduce the impact of the installation of the superstructure edge treatments.

These objectives reconfirmed that the use of Super-T beams, or other similar forms of long precast elements would not be suitable for this site, and verified the benefits of precast segmental construction.

During this review, the focus shifted from point of installation segmental construction techniques, to those that involve the full span erection of segmental structures.

Studies further identified that such a full span method would work best if a dedicated site assembly area was established within the project boundary, whereby factory type controls could be implemented and works could be predominately completed at ground level, with minimal impact on the adjacent residential properties and operating rail.

Through a rigorous and documented process of optioneering for various full span concepts and erection methods, a full span Straddle Carrier and Launching Beam method was developed by VSL and presented the best option in terms of programme, cost, interface with the rail corridor, and interface with follow-up construction activities.

VSL had previously used this type of equipment with success on a project in Taiwan for the construction of the Taiwan High Speed Rail (THSR). The key points of difference for the Caulfield to Dandenong project were the challenges associated with operations in a brownfield site in a live and narrow railway corridor, and the use of segments to form a span (full span on site casting was used for THSR).

Fig. 3 Straddle Carrier erecting a span for Taiwan High Speed Rail

The design of both the Straddle Carrier and Launching Beam and the permanent work involved a very high degree of coordination to ensure adequacy and compatibility of the independent designs.

Straddle Carrier

A Straddle Carrier is a self-propelled bridge crane mounted on tyres and designed to lift and carry a completed span. It drives on the previously erected sections of viaduct from an assembly and loading area to the location where the span is installed. The operational footprint of the machine is therefore constrained to the limits of the viaduct being erected.

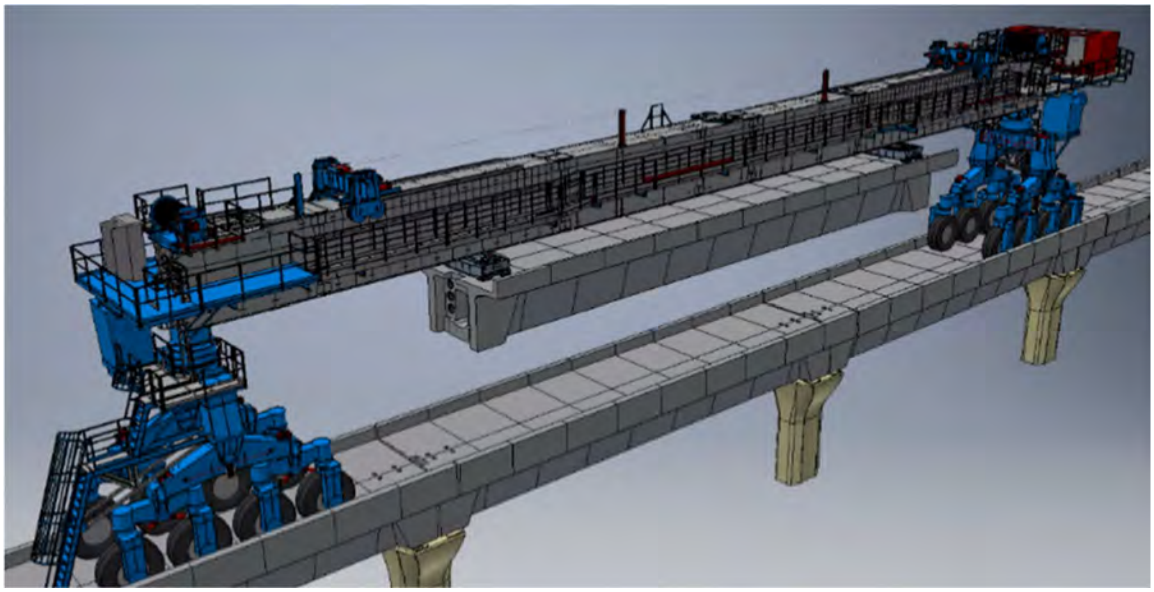


Fig. 4 Straddle Carrier on Proposed Viaduct

Launching Beam

In parallel to the Straddle Carrier operations, a steel box girder "Launching Beam" capable of spanning approximately twice the maximum span length is positioned at the erection front without intermediate support. The Launching Beam is used for to provide support for the Straddle Carrier as it is launched forward from the end of the completed section of viaduct, to position in readiness for the placement of the next span.

The Launching Beam was detailed to accommodate the vertical and horizontal alignment of the new structure, enabling the beam to operate at the variable gradients of +/- 2% and accommodate the curvature in plan (minimum radius = 1,000 m).

The Launching Beam system includes chains and chain pulling units to enable self-launch operation, thus once erected onto the viaduct the Launching Beam operates without additional plant.

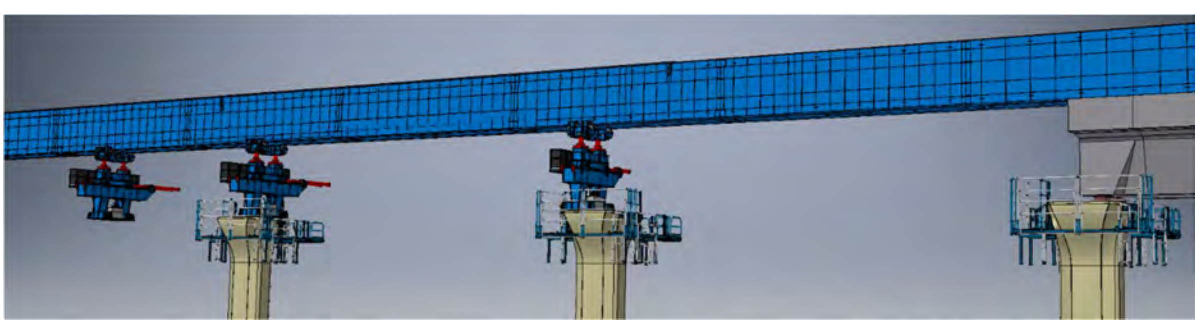


Fig. 5 Launching Beam on Proposed Viaduct

By using the Straddle Carrier and the Launching Beam, all the spans of the viaduct could be transported along the railway corridor to the point of installation, with minimal impact on railway operations and neighbouring infrastructure and properties.

iii. Construction Methodology

a. Precasting Operations (Pakenham Precast Yard)

A purpose built precast facility was established in Pakenham for the casting of the box girder segments. Field segments were match cast using the long-line casting technique which involves casting all segments within a particular span in their correct relative position on a fixed bed, with the side forms moving along the line as each segment is progressively cast. Fourteen long line casting beds were provided to meet the necessary segment production rates. Pier segments were significantly more complex due to the presence of the end diaphragm, tendon anchorages and access opening. These segments were cast on a separate bed using the short-line casting technique. Six short-line moulds were required to service the fourteen long line beds.

Fig. 6 Long Line Casting Beds at Pakenham Precast Facility

A total of 2226 segments were cast over a period of 13 months, utilising 25,000m3 of concrete and 6,000tonnes of reinforcement. Average production rates of 12 segments per day were achieved, peaking to a maximum of 16.

General Segment Casting

The following outlines the general sequence for casting of the segments:

1. Prefabricate 95% of the reinforcement cages on a slice table and in a jig.
2. Land the reinforcing cage in the match cast mould and cast the precast concrete box girder segments against its conjugate.

3. Open the forms when the concrete has reached the minimum compressive strength, clean & oil the side forms and relocate them longitudinally, using the segment recently cast as an end form.
4. Start again the cycle from 1.
5. A minimum of 3 segments of a same span are kept on the long line bed. Segments cast earlier, at the back of the production line, can be removed to be stored at the precast yard or installed in the short-line mould (+1 segments) to match cast the pier segments. Segment shall be lifted and stored after the concrete has reached a minimum compressive stress of 15MPa

b. *Assembly of Segments into a Span (Murrumbeena Assembly Area)*

A purpose-built Assembly Yard facility was established at Murrumbeena Station for the delivery of segments and the preassembly of each span.

The Assembly Yard set up included three Assembly Beds (each approximately 45m long) and three storage beds on which two spans could be stored.

Two number 230t overhead portal gantries supported on strip footings were erected for all handling operations, complimented by smaller cranes as required.

Fig. 7 Assembly Yard at Murrumbeena Station

On a typical 24 hour day, , the assembly yard would assemble and stress two spans, install the edge treatment and temporary handrails on two spans, grout two spans (stressed the day before), lift four spans (within the assembly yard) and lift the Straddle Carrier twice.

General span assembly procedure

The following outlines the general sequence for assembly of a span:

1. Deliver segments by semi-trailer and lift using one of the two 230 ton capacity Gantry Cranes,
2. Install the segment on an individual frame (these form part of the assembly bed system described in section 5.2.2.1),
3. Install ducting for external tendons and feed through strands,

4. Stress tendons in pairs (three pairs in total). The fully replaceable VSL Electrically Isolated Tendon (EIT) system was used to ensure protection against stray currents during operation.
5. Span lifting spreader devices are stressed onto the pier segments of the assembled span,
6. The edge treatments and handrails are installed on the span,
7. The span is lifted out of the assembly bed by the two synchronized 230 ton capacity Gantry Cranes and placed onto a storage bed. *The assembly bed can start the assembly of a new span*
8. The PT tendons of the span in storage are grouted using low bleed VSL Optimized mix,
9. Once the grout in the tendons has reached sufficient curing, the span is lifted onto one of the two viaducts by the two synchronized 230 ton capacity Gantry Cranes,
    a. The first 8 spans were erected directly onto the columns by the Gantry cranes.
    b. Once sufficient length of the superstructure was erected for the Straddle Carrier to operate, the overhead cranes were lifting the next assembled span on the bridge deck.
10. The Straddle Carrier is lifted by the two synchronized 230 ton capacity Gantry Cranes onto the span,
11. The Straddle Carrier lifts the span and drives it to its final location

To facilitate all lifting operations over the live rail line and platform operational platform, a system of heavy duty steel frames, capable of withstanding the impact of an accidental load release was implemented. Below these heavy-duty frames, smaller structures were installed to catch any debris from the construction activities.

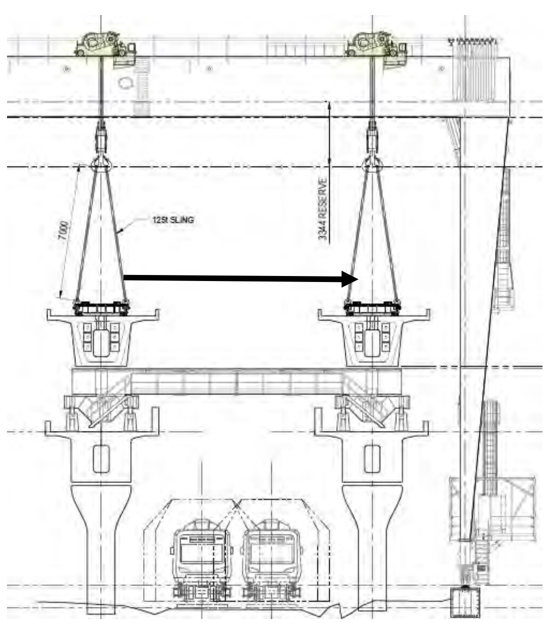


Fig. 8 Span lifted over live railway

c. Installation of each span

The Murrumbeena Assembly yard was situated in the middle of the project. Viaduct erection work fronts were serviced by one Straddle Carrier in conjunction with two Launching Beams.

General sequence for span installation

1. The laden Straddle Carrier reaches the Launching Beam,
2. The Straddle Carrier and Launching Beam are connected mechanically, electrically and hydraulically,

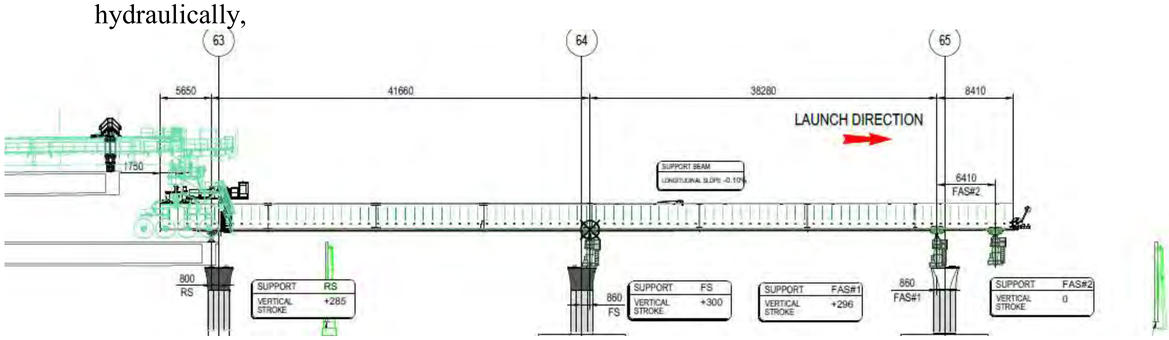


Fig. 9 Carrier Sequence Steps 1 and 2

3. The Straddle Carrier, loaded with the span, launches onto the Launching Beam,

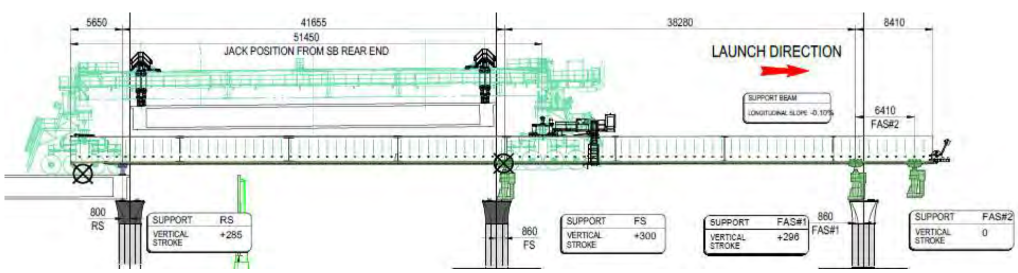


Fig. 10 Carrier Sequence Step 3

4. The two machines are disconnected
5. The Launching Beam launches forward to clear the span to be installed

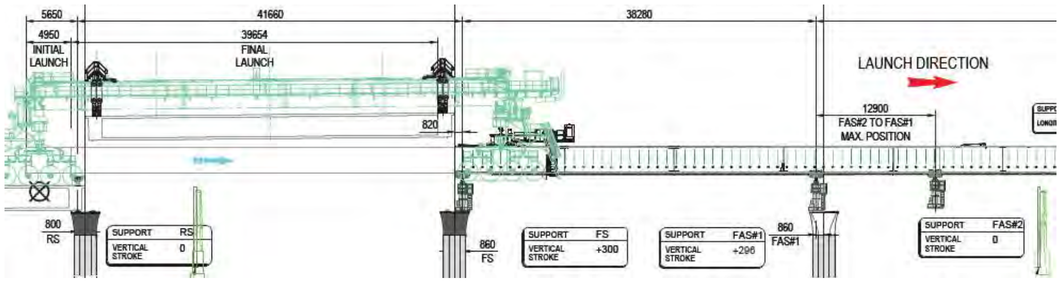


Fig. 11 Carrier Sequence Steps 4 and 5

6. The span is landed by the Carrier and adjusted onto its bearings,
7. The span lifting spreader beams are disconnected from the span and the Launching Beam is launched back.

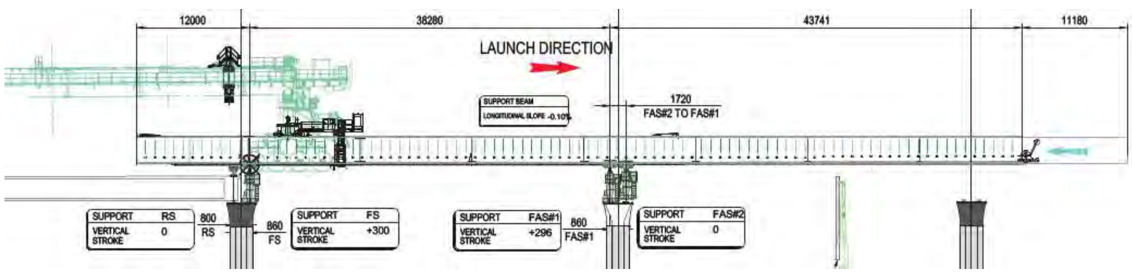


Fig. 12 Carrier Sequence Steps 6 and 7

8. The Straddle Carrier is launched back.

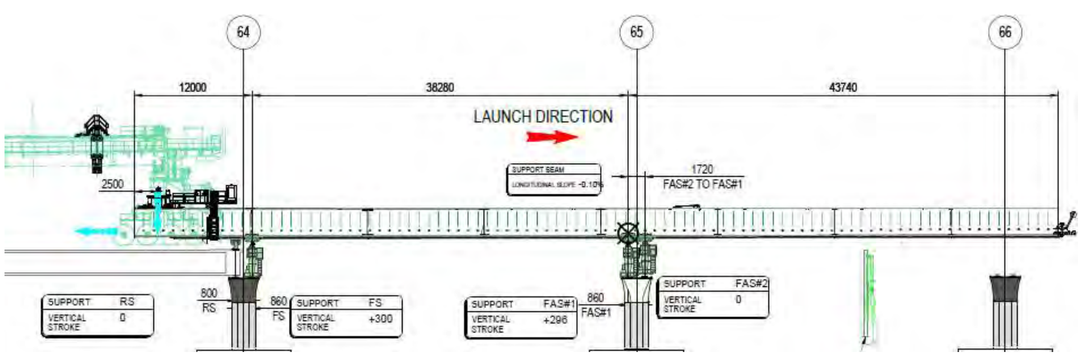


Fig. 13 Carrier Sequence Step 8

9. Straddle Carrier drives back on the viaduct to the Assembly Yard to be loaded with the next span.
10. During that time, the Launching Beam self-relocates in preparation for the erection of the next span when the Straddle Carrier returns.

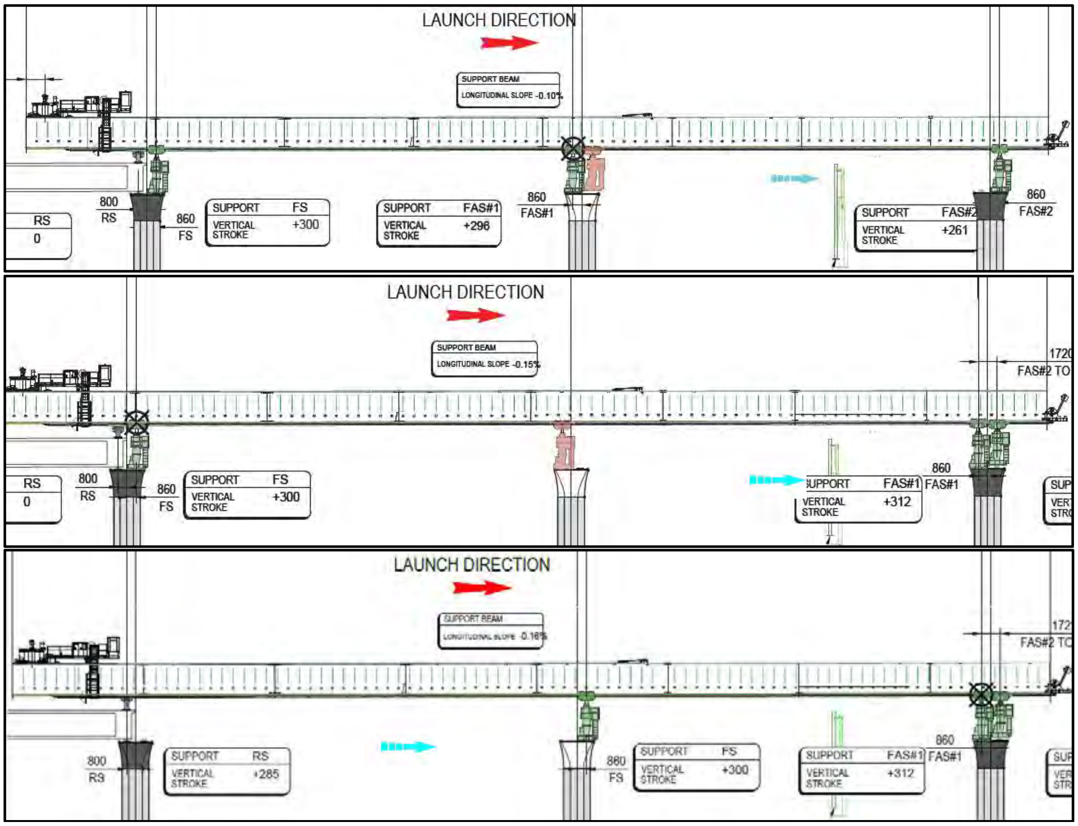


Fig. 14 Carrier Sequence Step 9 and 10

11. Once the superstructure was fully erected on that portion of the viaduct and the Straddle Carrier was operating on other sections of Area 1, the rest of the fitments were installed on the superstructure:
12. Casting of derailment kerbs ,
13. Overhead wiring structures, signal structures and any miscellaneous deck upstands installed.
14. Privacy screen/noise wall, architectural cladding, handrails and combined services route installed
15. Construction of the rail plinths and the installation of the rails

At peak production, 13 spans were installed in one week.

iv. Key Challenges and Innovations

a. Precast Yard

Reinforcing Slice Technique

The preassembly of transverse reinforcement slices was used to great success.
Reinforcing cages were detailed by the permanent work design team as a series of transverse slices of reinforcement. These were assembled together on a horizontal table and then rotated inside a jig, or template frame. The longitudinal lacer bars, bracing bars (if required), and any additional reinforcing including trimming bars were added inside the jig. The finalized reinforcing cage was lifted out of the jig and stored until the moulds would be ready to receive it.

This slice and jig technique enables the installation of temporary works to minimize manual handling, thereby improving the safety aspect of the works, and providing better access around the reinforcing cages.
The principles of mass production together with well-planned fundamentals for yard set up and temporary works greatly assisted in maximising productivity in the reinforcing area.

Fig. 15 Geometry of prefabricated reinforcing cage

Automated Segment Lifting Beam

Working at height is one of the major hazards when rigging loads. For this project, VSL adapted a patented segment lifting beam design, invented by its parent group company, Bouygues Construction. A mechanical system, operated by radio control, inserts the lifting arms of the spreader beam in to four penetrations cast in the top flange of the box girder. This eliminates the requirement to access to the top of the box girder to rig and disconnect the load.

The design of the mechanical part of the spreader was failsafe, such that a failure of the mechanical part of the lifting beam, in accidental conditions, would not have affected the safety of the load being handled.

The system considerably improved the safety of the segment handling.

Fig. 16 Patented Automated Segment Lifting Beam

b. Assembly Yard

Performing more than five heavy lifts per day

The assembly area was serviced by two 230 ton capacity Portal Gantries, to perform daily within a 24 hour period, more than:

- 30 segment lifts (20 to 45 ton),
- four spans lifts (300 to 430 ton),
- two Straddle Carrier lifts (265 ton),
- and all the minor lifts (ranging from 5 to 15 ton),

The role of these gantries was to perform all lifting operations without the need for additional plant. They were detailed with a serviceable height of 25m and clear span of 40m to provide capacity to operate over the full width of the construction footprint at Murrumbeena Station.
The self-propelled gantries were supported on two 156 m long continuous strip footings and could synchronize to perform tandem lifts while being operated by one remote control.

Despite the size of these large cranes, a considerable number of lifts were planned and operated with only 100mm clearances. Many factors had to be considered during the design and preparation of the activities, including: the geometry of all coexistent plant & temporary work; the deflection portal cranes at different position of the lift; and the estimated elongation of slings.

Fig. 17 The two number 230 tonne Capacity Gantry Cranes in the Assembly Yard

The Assembly Bed and Stressing Operation

To avoid the complexities of epoxy application adjacent to the live rail, a dry joint system was incorporated into the final design of the span. This also avoided the need for temporary prestress, including blisters or equivalent. As a result, a specialised assembly bed was developed to allow for the controlled movement of segments (and closing of segment joints) during the activation of the permanent post tensioning. The span assembly bed was detailed such that segments were landed on independent frames with longitudinal sliding capabilities. These frames were fitted with small vertical hydraulic jacks for the controlled support of the segment being placed.

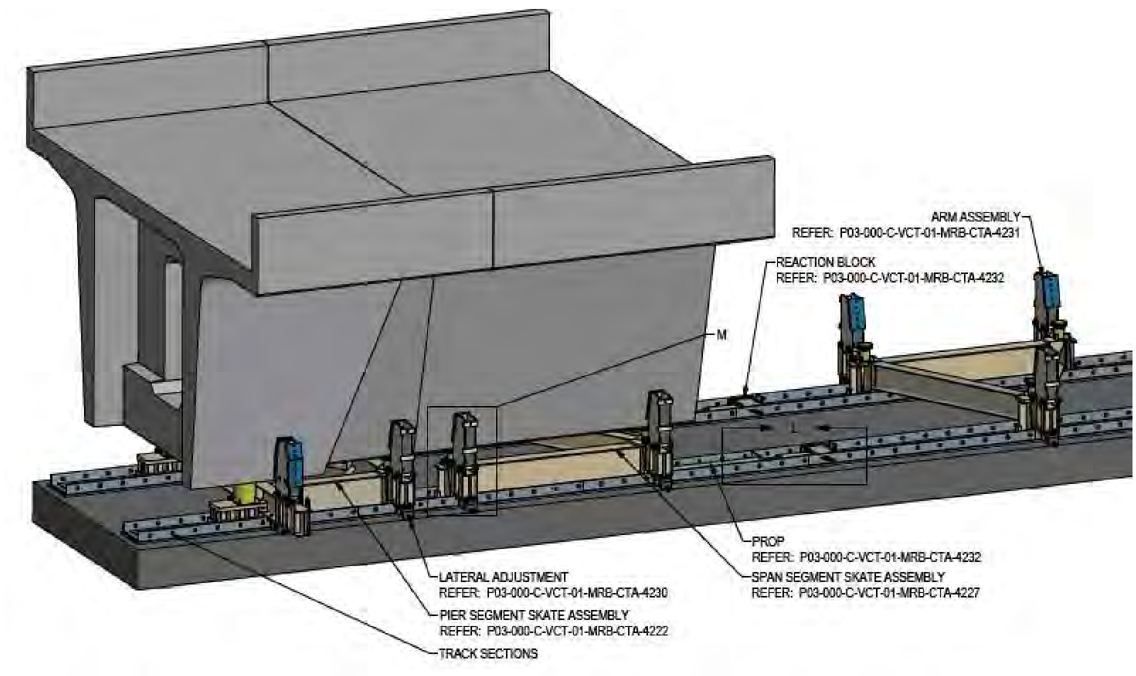
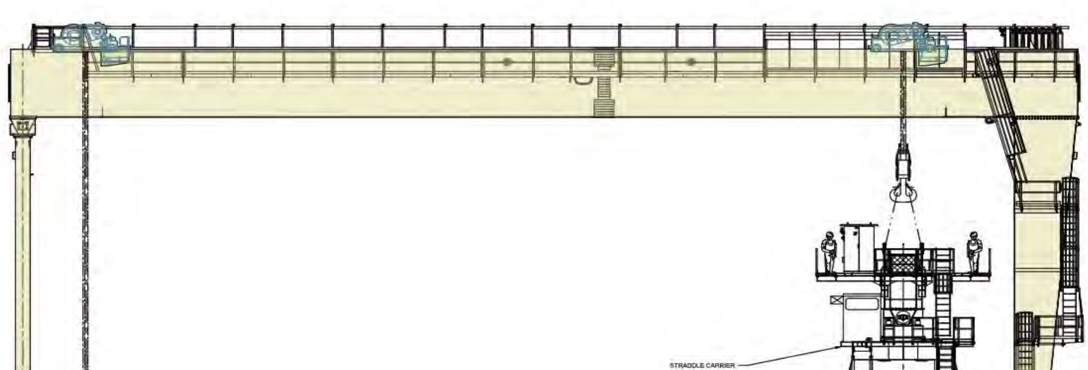


Fig. 18 Isometric View of Span Assembly Bed

The frames receiving the pier segments at each end of the span were of a different design. During the installation of the segments on the assembly bed, each segment was supported by its own frame. During the stressing operation, the formation of the upward deflection of the span gradually transferred the vertical reactions to the two outer frames. Therefore, the pier segment support frames were designed with four points of support; two of which had the capacity to support half the self-weight of the full span. To prevent unequal transfer of the load, the four hydraulic jacks on each outer frame were linked to provide a three point support system. A similar hydraulic connection was set up between the jacks supporting the full span to maintain equal loading between the supports.

The Assembly Bed and Stressing Operation

Spans were double stacked on three storage beds adjacent to each other.
Each bed had the capacity to sustain the 800t+ load. The interface between the lower span and the beds was achieved on one end of the span by two vertical hydraulic jacks and on the other end of the span by high capacity elastomeric bearings. The jacks were linked hydraulically during the load transfer and then secured using a locknut.



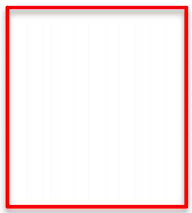


Fig. 19 Cross Section of Murrembeena Assembly Yard, Highlighting Storage Bed

Transfer Beams for the Protection of Murrumbeena Station and the Live Railway

A set of safety platforms and debris screens were developed to provide protection to the fully operating station and rail infrastructure below during the operation of the gantries. These safety platforms, or "transfer beams", were designed to support the accidental release of a full span (430 tonne), or the Straddle Carrier (265 tonne) as these were relocated over the rail lines to the up line (southern) track.

The Transfer beams consisted of two 85t steel frames detailed with an integral shock absorption system to reduce the impact effects of an accidental load release.

The maximum fall height was limited to 100mm, which limited the impact load to the equivalent of five times the self-weight of the span

The transfer beams were also designed with a longitudinal sliding rail system that enabled them to be relocated before each lift and to suit the length of the item being lifted.

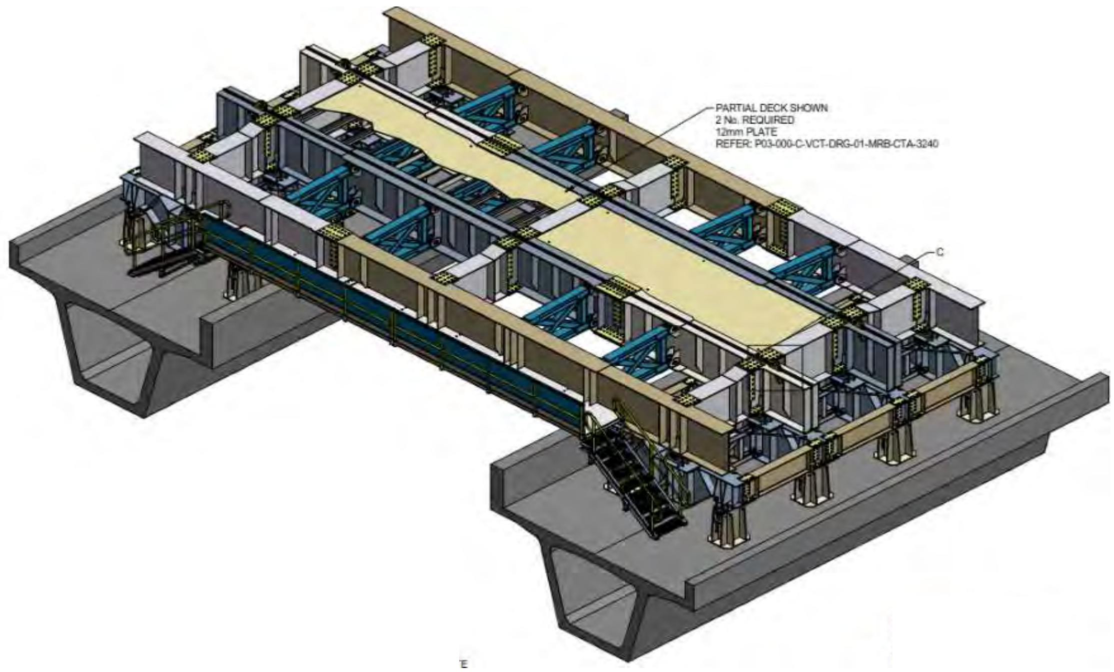


Fig. 20 One of two Transfer Beams (sliding system not shown)

c. Erection of Spans

Straddle Carrier and Launching Beam

As the Straddle Carrier operated at deck level, traveling along previously erected spans and imposed large loads on the deck, the top flange cantilevers of the boxes were thickened to accommodate the Carrier. The Carrier operated at low speeds to reduced dynamic impact with movements along the bridge deck restricted (for permanent work design assessment) to a maximum transverse eccentricity of +/- 300mm. The geometrical clearances of the span, with its fixtures, rarely enabled the Straddle Carrier to deviate more than +/-200mm.

The Straddle Carrier was able to operate in both directions. The Launching Beam was only designed to operate in one direction.

There were two operators in the Straddle Carrier (one in each cabin, either end of the Straddle Carrier) and it could drive in manual, semi-automated or fully automated mode.
The automated mode relied on a guidance system, based on an induction wire attached to the deck. This automated guidance system maintained the path of the Straddle Carrier within the permissible eccentricities detailed above.

The Straddle Carrier and Launching Beam could potentially attract high wind loads due to the large and vertically eccentric sail area and drag coefficients. Normal span delivery and erection operations of the Straddle Carrier, were subject to a well-defined set of rules defined in a specific wind procedure. The procedure acted as a guideline for the engineers and supervisors to decide when the machine could be operated or not.

The implementation of safe access and the ability to rescue someone at any location on the machine was successfully incorporated into the design of the equipment.

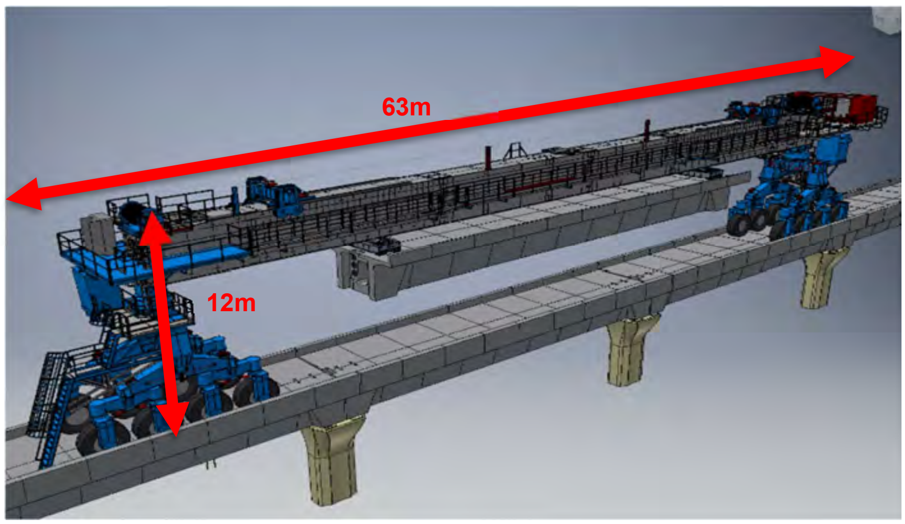


Fig. 21 Key Dimensions of Straddle Carrier

Key straddle carrier characteristics:

- Designed & fabricated specifically for this project
- Self-weight of the machine: **265 tonnes**
- Rated Lifting Capacity: **4,270 kN**
- Max travelling speed (loaded with span): **2.5 km/h**
- Max travelling speed (unloaded): **5 km/h**
- Max steering angle: **5°**
- Max gradient for operations: **2%**
- Max lifting / lowering speed: **2 m/min**
- Range of span lengths the straddle carrier can lift: **26.9 m to 39.9 m**
- Transverse adjustment of the Upper Cross Beams: **+/-200mm (to accommodate centre of gravity offset)**
- There is a generator mounted on the top of the Straddle Carrier. The generator is refilled at high level at Murrumbeena station.

Fig. 22 Key Dimensions of Launching Beam

Key Launching Beam characteristics:

- Designed & fabricated specifically for this project
- Steel Box Girder composed of 8 elements (in longitudinal direction) with bolted connections.
- Self-weight of the Launching Beam including all the equipment: **255 tonnes**
- Maximum cantilever during operations: **59.14m (recorded deflection of 485mm at that distance)**
- Each Launching Beam was supported by a minimum of two legs and a maximum of three legs
- Each had four legs in total (three standing on the piers and one standing on the deck)

Transporting and erecting spans in close proximity to an operating railway

As the project was the first of its type, the definition of clearance requirements to the rail safety zone was developed during the detailed design phase of the permanent works. These minimum requirements were defined through close collaboration of the Alliance partners and included all aspects of the temporary staging required for the works.

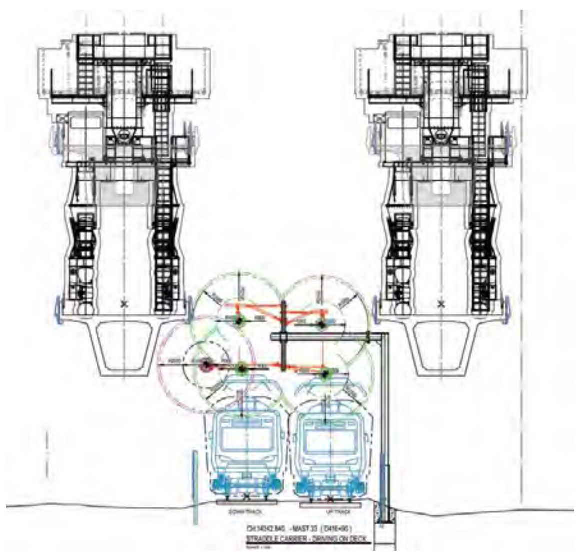


Fig. 23 Cross Section produced to define the Clearance Requirements to the Railway Corridor

Placement of the Precast Span onto the Supports

It was anticipated that the levels of the four points on the soffit of the span would see some variations in levels where it is supported. Because the spans were cast in two types of moulds (long-line for the field segments and deviator segments; short-line for pier segments), the assembled span could have some minor variation in relative level between the four points. Although the span was being stressed symmetrically by pairs of tendons at a same time, a very slight twist of the span would often occur during the stressing. The overall magnitude of the two phenomena was approximately 1 to 2 mm in level difference at one corner of the span. Due to the construction methodology, it was difficult to associate a value to each action.

Elastomeric bearings are at the interface between the precast piers and the isostatic superstructure. The bearings were observed to have fabrication thickness tolerances of +/- 2 mm.

The bearings were laid on grouted pedestals to account for the finished level of the precast piers. The grout pedestals on which these bearings were set also had a tolerance of construction of +/-1 mm.

The combination of all these tolerances, if disregarded, could lead to a theoretical variation of level at one support of approximately 5 mm.
The stress-strain properties of the elastomeric bearings indicated that the bearing itself could absorb approximately 1.5 mm theoretical variation under the loads considered. Above that value, adverse additional loads would be transferred to the lower supports.
The design of the deck could accommodate this additional load. However, the piers and bearings were found to be over-loaded under the laden Straddle Carrier and a 5 mm variation, in the level of one support relative to the plane generated by the three other points, was not acceptable.

With a traditional span-by-span erection method, the span is often landed on temporary bearings so that the grouting of the underside of the bearings and the fixing of the span geometry is off of the critical path.

The Launching Beam system has support points on the pier tops at the exact location where the bearings are to be installed. Therefore, the pedestals had to be cast and cured before the Launching Beam would land its leading leg on to the pier, approximately two piers ahead of the span being erected. As most of the spans would be erected within 48 hours after being stressed, there was very little time to calculate the required pedestal heights and cast sufficiently in advance to achieve the required strength.

v. Permanent Works Design

a. Impact of the Straddle Carrier on the Viaduct Design

The permanent works design of the viaduct was accelerated with respect to the design and fabrication of the Straddle Carrier system, as this was critical for early procurement of segment moulds and setup of the precast yard in Pakenham. Segment geometry was fixed at the preliminary design stage, more than a year before design completion of the Straddle Carrier system. At this stage of the project, the viaduct was intended to be erected span-by-span with a conventional overhead gantry crane. Span erection with a Straddle Carrier system only became a real possibility after the preliminary design submission, and after the sub-alliance partner, VSL, had developed sufficient confidence with the system and its use within the constrained rail corridor.

Preliminary loads for the Straddle Carrier wheel groups and Launching Beam support legs were provided by the supplier, based on Straddle Carrier systems which they had previously designed for similar projects. Basic operational phases and kinematics of the system were also defined. Key design parameters including geometry, mass, distribution of mass, windward area and aerodynamic drag coefficients were obtained from the supplier. This enabled the permanent works design team to undertake an independent assessment of the Straddle Carrier and its operations.

Use of the Straddle Carrier system places large concentrated loads on the bridge deck due to span delivery, erection and launching phases. In addition, the Launching Beam rear support leg is located on the pier segment of the previous span, with the front and front auxiliary support legs positioned on the next crossheads immediately beyond the working front. The original erection scheme, with an overhead gantry, did not place any concentrated wheel loads on the bridge deck and gantry support leg loads were significantly lower in magnitude. Consequently, the segment geometry was modified to accommodate these new loads and the moulds amended to suit. Top flange haunches at the web interface were increased by 100mm but this pushed the overall span weight beyond the capability of the Straddle Carrier system. The design of the top flange beyond the derailment kerb and the web thickness at the deviators were refined to ensure there was no net increase in span weight.

For the purposes of design, it was initially planned to consider the complex construction equipment as equivalent to a "heavy platform load" as outlined by the Australian Bridge Code, AS5100. However, this would produce an overly conservative design and lead to significant changes to the box girder design. The heavy platform load is intended to cover slow moving vehicles with heavy loads. Relevant load factors are conservative for this vehicle because it is an idealized 'design vehicle', to be used when there is uncertainty associated with load type, axle configuration, mass and the distribution of mass.

The Alliance recognised that many measures and procedures were implemented to remove uncertainty associated with the Straddle Carrier and its operation and an ultimate limit state load factor of LF=1.2 was adopted for all Straddle Carrier and Launching Beam loads for laden and unladen cases, justified by the following points.

- The actual mass of the Straddle Carrier and Launching Beam is known. Both elements are constructed from steel and their mass has been confirmed by the manufacturer. This includes all primary and secondary steelwork, motors, mechanical and electrical equipment. The Straddle Carrier is also lifted by the gantry cranes setup at Murrumbeena station. These are fitted with load cells which allows the weight of the Straddle Carrier to be verified
- The mass of spans erected by the Straddle Carrier system is known. The overhead gantry crane at the Pakenham precast yard is fitted with load cells and the mass of each precast segment is recorded prior to delivery to site. The weight of the assembled and stressed spans is also verified by the overhead gantry cranes at Murrumbeena Station. The Straddle Carrier is rated with a safe working load and its payload cannot exceed the 430t limit during its operation
- The Straddle Carrier represents a moving load with a defined mass. Movements of the Straddle Carrier are at slow speed and the system is operated in a controlled manner. Operations of the Straddle Carrier are covered by an extensive construction procedure. Design loads provided by the manufacturer are inclusive of a dynamic load allowance. This was taken as $\alpha=0.1$ following rigorous analysis by the manufacturer and is similar to the value for the heavy platform load.
- Straddle Carrier operations are limited to wind speeds less than 72km/hr. When laden with a span, the Straddle Carrier has a large eccentric windward area. This produces an overturning moment on the span and significant transverse bending in the piers. Weather forecasts were reviewed by the construction team prior to operation. When storm conditions are forecast, the tie-down procedure is implemented whereby the payload is lowered to deck level and secured. The permanent works design includes shear pins and an uplift restraint device. These are installed immediately after span erection and engaged during Straddle Carrier operations
- The Australian Bridge code permits relaxation of self-weight load factors for precast segmental construction where appropriate control and monitoring is exercised over dimension and mass. A load factor of LF=1.1 was considered for dead load of previously erected spans

It is important to note most of the viaduct design was based on preliminary Straddle Carrier loads and this presented significant risk to the project. This is typical for projects where specialised construction equipment is used and its operations govern design. A detailed assessment was undertaken by the permanent works design team at each stage of the Straddle Carrier design process. This was imperative in identifying the risks associated with the Straddle Carrier design, managing any changes, and understanding the complex interaction with viaduct.

Typical piers consist of precast segments which are post tensioned together with high-tensile bars to form a column. Joints between segments are unreinforced and the design relies exclusively on the compression provided by the bars. For the permanent design, ‘zero tension’ was permitted across segment joints under all serviceability limit state load combinations. During construction, the Straddle Carrier and Launching Beam place columns in a state of biaxial bending and this was found to govern the pier design. The ‘zero tension’ requirement across segment joints was relaxed under particular construction load combinations where the case only occurs once, the magnitude of tensile stress is small, and the area of the joint experiencing tension is localised. This only occurred during span launching phases where the Straddle Carrier is mounted on the Launching Beam. Significant longitudinal and transverse moments are generated in the pier due to: the transverse eccentricity of the Launching Beam and Straddle Carrier; and the longitudinal eccentricity of the support legs, relative to the pier centreline. After the span is landed on the permanent bearings, the column returns to a state of residual compression.

Fig. 24 Critical Launching Phase for the Piers

A series of portal frame structures were designed where the existing tracks slew beneath the new viaduct. At these locations, conventional piers could not be used to directly support the viaduct as construction activities would require a large number of rail occupations and prolonged closures. A pier with an eccentric column was also investigated however discarded early in the design. It required a deep corbel to the support the viaduct and this led to clashes with the existing overhead and catenary lines. The tender design adopted a concrete portal beam made integral with its columns. Once clearances to the existing overhead and catenary lines were confirmed by survey to be tight than those from the tender design, and requirements for the Straddle Carrier and Launching Beam system were understood, it was evident the concrete solution would not work. The Launching Beam and its support legs are setup relative to top of crosshead level for the standard piers. Top of portal beam level was set higher to clear the existing overhead and catenary lines, and is arrangement was not compatible with the support legs. The overall depth of the portal beam was restricted to 900mm below top of deck level to avoid major changes to the design and fabrication of the Launching Beam. The final solution involved the use of a steel portal beam made integral with its columns. A two stage insitu deck was designed. The first stage was designed to act composite with the steel portal beam and accommodate loads from the Launching Beam support legs. The second stage was built up to top of deck level to support the tracks and other bridge deck furniture. This stage was cast after Straddle Carrier had passed. These requirements added significant complexity to the design and construction. In addition to this clearance to the existing overhead and catenary lines prevented elastomer bearings being placed directly beneath the webs of the box girder. Concrete outriggers were designed for the pier segment to accommodate placement of the bearings beside the web. These were cast as a second stage pour at the precast yard to avoid changes to the standard pier segment mould. Transverse post tensioning was used to connect these outriggers to the box girder core and control cracking.

Fig. 25 Typical Portal Frame and Concrete Outriggers

b. Flexural Capacity

A finite element analysis was undertaken to determine the behaviour of the precast segmental box girder beyond decompression. At service limit state the entire structure is designed to remain under a minimum compressive stress of 1 MPa and exhibits linear elastic behaviour until decompression is reached. External tendons have been adopted for the viaduct to reduce span construction cycle times. Tendons are external to the surrounding concrete and are contained within plastic ducts. Tendons are unbonded to the surrounding concrete and only come into contact with concrete at anchorage points located at each pier segment and at the deviators. All tendons are fully replaceable. As strains between the tendon and concrete are not compatible, the tendon force only increases due to global deformation of the entire span. This behaviour is substantially different to a conventional monolithic bridge girder. Beyond decompression, segment joints begin to open and overall span stiffness rapidly decreases. At the ultimate limit state, joints near mid-span are anticipated to open and the overall structure will exhibit large deformations with highly non-linear behaviour. Match cast dry joints must be capable of transferring shear forces from flexure and torsion to ensure the flexural capacity is not limited by joint failure. This is important at mid-span where joints are subject to relatively low shear and torsion however open over a significant depth. The Midas FEA software package was used to develop a three dimension model of typical box girder spans. Each span consists of pier, deviator and field segments. Geometry specific to each segment type was considered. Concrete segments were represented with solid elements. Tendons were modelled as truss elements following the three dimension design profile. Tendons are external and connected to concrete elements only at anchorage points at pier segments and deviators. All segment joints are match-cast dry type joints. These have been modelled with face contact elements to account for segment joint opening. These joints do not have any tensile load carrying capacity. Shear keys at segment joints are represented with rigid links to permit vertical shear transfer across joints.

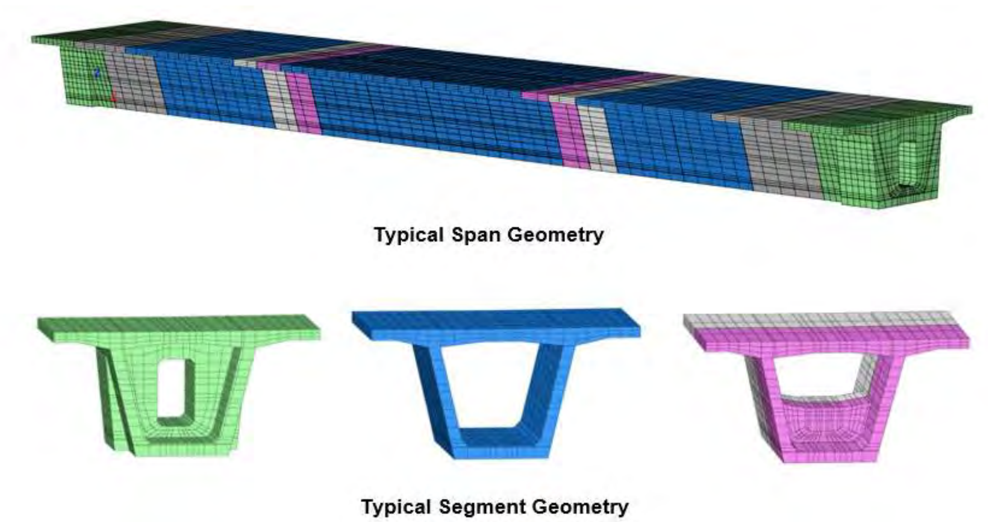


Fig. 26 Typical Span and Segment Geometry

The analysis considers the following:

- Opening of match cast dry joints beyond decompression without tensile load carrying capacity
- Non-linear material properties of concrete and steel
- The segmental box girder is detailed without a deviator at midspan. As the structure decompresses and segment joints begin to open, the span undergoes significant deflection. The analysis accounts for the loss of tendon eccentricity under large span deformations
- At ultimate limit state the tendons may slip relative to the deviator if friction between the tendon and surround concrete is overcome. Tendons are connected at the deviator with rigid links. These links permit relative longitudinal movements between the tendon and concrete however restrict vertical and lateral movements. A sensitivity analysis of deviator friction and the effects on flexural capacity has been carried out to assess the possibility of significant friction developing, without slip.

The ultimate flexural capacity was deemed to be reached when one of the following failure criteria was reached:

- Concrete crushing in the extreme compression fibre of the box girder top flange. The maximum principal compressive strain was limited to $\varepsilon c = 0.003$ mm/mm
- Tendon failure where the tendon stress exceeds the ultimate tensile strength of tendon fpu = 1860 MPa

The ultimate flexural capacity of the box girder was taken as the lowest applied moment corresponding to either of the above failure criteria. A strength reduction factor of $\varphi=0.80$ was then applied in accordance with AS5100.5 [1]. The analysis has revealed flexural capacity of the precast segmental box girder is sensitive to tendon configuration, span segmentation, and the proximity of the deviator segments relative to mid-span. Spans with an even number of segments with a joint coinciding at mid-span performed poorer than spans with an odd number of segments. Joint opening for these spans was found to be largely concentrated to a single joint. Spans with closely spaced deviators performed better than those spaced further apart with the loss of tendon eccentricity being less pronounced. All spans exhibited concrete crushing failure in the box girder top flange prior to tendon failure. Refer below for a summary of findings for critical span types.

Table 1. Flexural Capacity Summary

Span Type	Girder Length (m)	Flexural Capacity ØMu (kNm)	Design Moment M*ULS (kNm)	Increase in Tendon Stress Δσp (MPa)	Joint Opens (No.)	Opening Depth (mm)
Non Station	39.9	71,840	65,500	330	3	0.35D
Station	28.9	70,920	47,900	600	0	N/A
Station	26.9	68,600	41,500	560	0	N/A

Notes:

1. The increase in tendon stress is based on the average increase of all tendons.
2. The maximum joint opening depth and the number of joints open within the overall span under the design ultimate limit state bending moment.
3. The opening depth is expressed as a ratio of the overall box girder section depth.

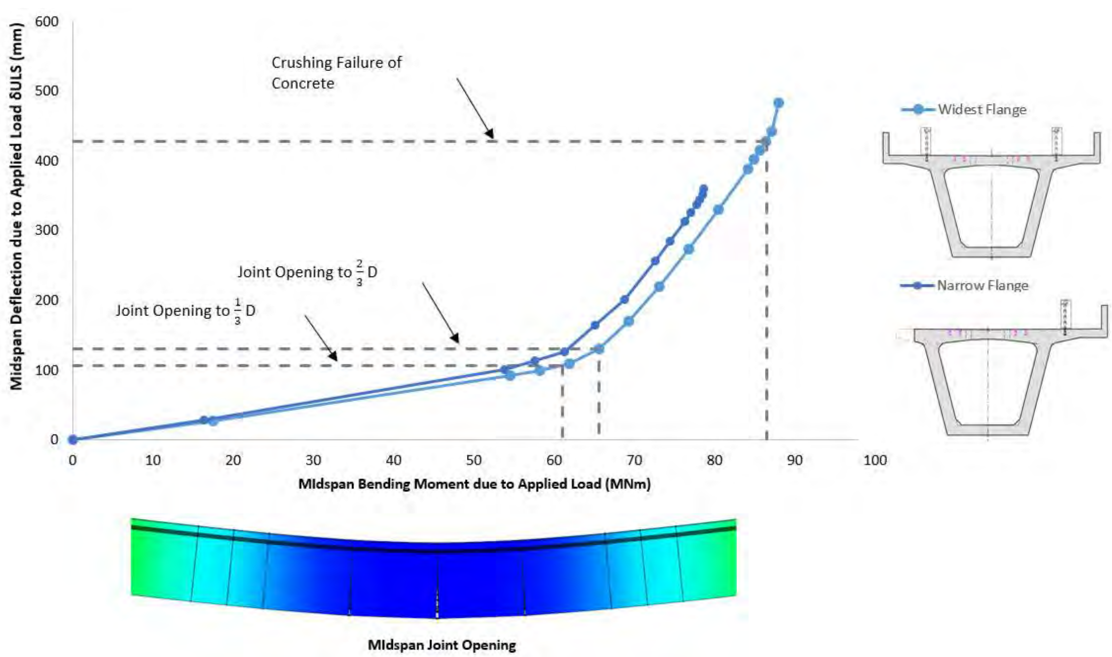


Fig. 27 Moment-Deflection Curve for the Non Station Spans – 39.9m

c. Match Cast Dry Joint Capacity

The precast segmental box girder utilises match cast dry joints. This form of joint was adopted to reduce span construction cycle times by removing the need to install epoxy across the joint prior to the stressing operation. Segments are match-cast with a series of shear keys formed into each web and flange. This type of bridge construction has been employed worldwide with extensive use in America, South East Asia and Australia.

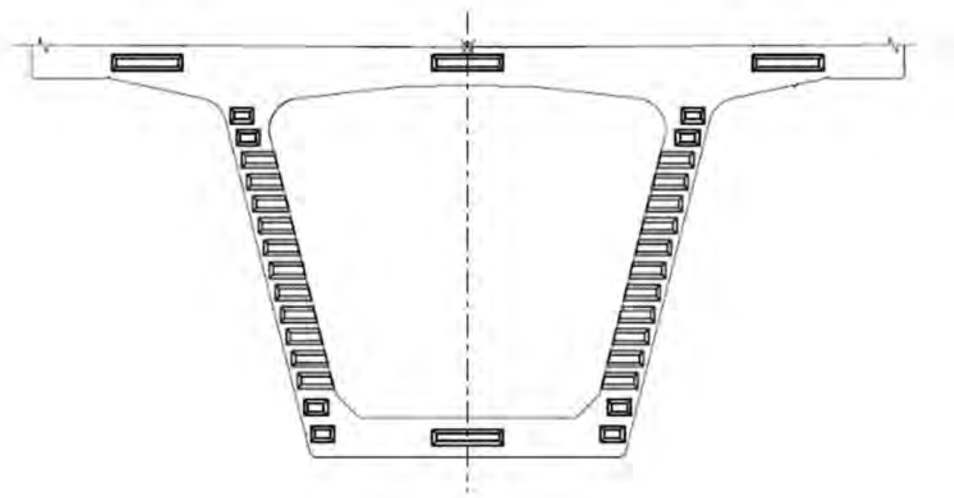


Fig. 28 Typical Match Cast Dry Joint

In recent years, the recommendations of AASHTO have moved away from the use of match-cast dry joints in precast segmental bridges. It now requires that all joints to be either cast-in-place closures or match-cast joints with epoxy to improve tendon durability in areas subject to freezing and thawing cycles or de-icing chemicals. AASHTO LRFD 2012 no longer provides guidance for the design of match cast dry joints in precast segmental bridges. The previous edition of this standard is used for the design of these joints: AASHTO Guide Specification for Design & Construction of Segmental Concrete Bridges, 2nd Edition 1999 [2]. Although dry joints are not permitted by AASHTO, bridge structures in this project are located in a temperate inland environment not subject to freezing and thawing cycles or de-icing chemicals. Environmental conditions in Australia are favourable for the use of match cast joints.

Joint capacities based on AASHTO guidelines have been compared against other international standards and recognised technical publications to ensure they are not overly un-conservative. Recent studies carried out by Rombach [3] and Shamass et al. [4] indicate the shear capacity predicted by AASHTO is un-conservative at high compressive stresses due to an overestimate in friction. Both studies recommend a reduction of the friction coefficient for joints subject to high compressive stresses. The principles and recommendations from these international standards and technical publications have been used to determine joint capacity at ultimate limit state for comparison. A strength reduction factor of φ=0.70 was then applied in accordance with AS5100.5.

All methods recognise shear capacity of precast segment joints is a combination of the friction developed across smooth and keyed regions due to compressive stresses across the joint, and the direct load bearing capacity of shear keys along the joint. The load bearing capacity of shear keys is dependent on concrete tensile strength and the area of shear key across the failure plane.

Table 2 Shear Capacity Summary of Closed Segment Joints

Span Type	Girder Length (m)	Design Shear VULS* (kN/web)	AASHTO 1999 ØVnj (kN/web)	Rombach 2002 ØVd,j (kN/web)	EN1992-1-1 2004 ØVRdi (kN/web)	AS5100.5 Interim ØVu (kN/web)
Non Station	39.9	2500	6825	5075	3845	4020
Station	28.9	2850	5975	4500	3255	3360

Notes:

1. Design shear includes vertical shear and the shear produced by torsion
2. The above capacities are based on the segment joint closest to the pier subject to the highest combined shear and torsion actions. These joints are in a state of residual compression and do not open at the ultimate limit state.
3. Design shear forces include the vertical component of post tensioning force due to the inclined tendon profile between pier and deviator segments. For simply supported spans this acts in the opposite direction to the applied design loads
4. Assessment to Eurocode EN 1992-1-1:2004 Clause 6.2.5 is based on roughness factors for an indented surface
5. Assessment to AS5100.5 Interim Clause 8.4.3 is based on a deliberately roughened surface with shear keys

Segment joints within the viaduct are subject to high compressive stresses from post-tensioning forces and the above summary highlights AASHTO overestimates joint shear capacity. The design of match cast dry joints has been carried out with a reduced friction coefficient in accordance with the recommendation outlined by Rombach (2002).

The Australian Bridge Design Code AS5100.5 Interim provides two methods for the assessment of joint shear capacity. The shear friction concept in accordance with Clause 8.4.3 has been adopted for comparison against AASHTO as this is similar to the method adopted by Eurocode. An alternative procedure is provided in Appendix B4 specific for dry joints. The strength contribution from shear keys along the failure plane is based on concrete tensile strength with marginal enhancement from the effect of compressive stress. In addition, a lower friction coefficient is used for smooth regions when compared against other methods. These modifications result in an overly conservative capacity.

At the service limit state all joints are designed to remain under residual compression without joints opening. Beyond decompression segment joints begin to open and this leads to a loss of contact as match cast dry joints do not possess any tensile load carrying capacity. The shear capacity of segment joints is reduced when the joint opens because the contact area across the joint is reduced and fewer shear keys are engaged. The capacity is sensitive to the depth over which opening occurs, this is based on finite element analysis results. At the ultimate limit state, most segment joints remain closed, particularly those near supports subject to high combined shear and torsion. Only a small number of segment joints near mid-span would open but these sections are subject to low combined shear and torsion. Torsional stresses are influenced by the extent of joint opening. Segment joints under residual compression are capable of transmitting torsion via shear flow around the closed perimeter of the box girder core. When segment joints open, this flow path is interrupted and torsion is transmitted as shear flow around the open section. These effects have been considered for joint shear capacity.

Table 3 Shear Capacity Summary of Open Segment Joints

Span Type	Girder Length (m)	Design Shear V* (kN/web)	AASHTO 1999 ØVnj (kN/web)	Rombach 2002 ØVd,j (kN/web)	EN1992-1-1:2004 ØVRdi (kN/web)	AS5100.5 Interim ØVu (kN/web)
Non Station	39.9	1445	4830	3515	2420	2645
Station	28.9	All joints remain closed				

Notes:

1. Design shear includes vertical shear and the shear produced by torsion
2. Above capacities are based on segment joints at mid-span subject to relatively low combined shear and torsion actions. These joints open at ultimate limit state.
3. Design shear forces include the vertical component of post tensioning force due to the inclined tendon profile between pier and deviator segments. For simply supported spans this acts in the opposite direction to the applied design loads
4. Assessment to Eurocode EN 1992-1-1:2004 Clause 6.2.5 is based on roughness factors for an indented surface
5. Assessment to AS5100.5 Interim Clause 8.4.3 is based on a deliberately roughened surface with shear keys

vi. Conclusions

The removal of the four existing level crossings between Caulfield and Hughesdale with elevated viaducts mitigated the high risks associated with the relocation of gas, electricity and water supply services and avoided compulsory land acquisition. The use of a specialised form of the precast segmental construction technique allowed most of the works to be completed offsite with reduced impacts on the local community. The design and construction techniques employed allowed the viaduct to be built without a large number of rail occupations and road closures, despite the majority of works being carried out adjacent to live rail. The unique Straddle Carrier and Launching Beam system met and exceeded production expectations with up to 160m of viaduct installed in one day, even though this was the first application of such a scheme in Australia.

References

[1] AS5100.5 Interim Bridge Design Code Part 5 – Concrete

[2] AASHTO Guide Specification for Design & Construction of Segmental Concrete Bridges, 2nd Edition 1999

[3] Rombach, G., “Precast segmental box girder bridges with external prestressing”, INSA Rennes, 2002

[4] Shamass, R., “Finite-element analysis of shear-off failure of keyed dry joint in precast concrete segmental bridges”, ASCE, 2014.

[5] EN1992-1-1:2004 Eurocode 2 - Design of Concrete Structure