VIADUCTS IN THE ARMADALE LINE UPGRADE PROJECT

Viaductos en el proyecto de mejora de la línea Armadale

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ABSTRACT

The Victoria Park - Canning Level Crossing Removal project is the first rail viaduct project constructed in Perth, Western Australia. The project stretches over 7 km on the Armadale Line and comprises four sections of elevated rail structures with a total length of 4.6 km, one principal shared path bridge for active transport, and five elevated stations. The superstructure of the elevated rail structures consists of two precast L-shaped elements joined by an in-situ slab in lower flange, to form a U-shaped trough girder. The design development of the U-troughs, software challenges discovered during the internal verification and integration of rail related geometry, services and egress are discussed in this paper.

RESUMEN

El proyecto de eliminación de cruces a nivel de Victoria Park - Canning es el primer proyecto de viaducto ferroviario construido en Perth, Australia Occidental. El proyecto se extiende a lo largo de 7 km en la línea Armadale y comprende cuatro tramos de estructuras ferroviarias elevadas con una longitud total de 4,6 km, un viaducto principal de uso compartido para transporte activo y cinco estaciones elevadas. El tablero de las estructuras ferroviarias elevadas consiste en dos elementos en forma de L prefabricados unidos por una losa in situ por el ala inferior, para formar una viga en forma de U. En este documento se describe el desarrollo del diseño de los tableros en U, los retos técnicos de software descubiertos durante la verificación interna y la integración de la geometría relacionada con el ferrocarril, los servicios y las salidas.

KEYWORDS: Precast concrete, light rail transit, prestressed concrete, U-shaped section, rail viaduct.

PALABRAS CLAVE: Hormigón prefabricado, tránsito ligero sobre raíles, hormigón pretensado, sección en forma de U, viaducto ferroviario.

1. Introduction

The Victoria Park - Canning Level Crossing Removal Project involves the removal of level crossings on the inner section of the Armadale Line, which stretches over 7 km and comprises four sections of elevated rail structures with a total length of 4.6 km. All works are limited to the existing rail corridor, elevating the two electrified rail tracks to improve public transport safety, create new and versatile public open space for the community and reduce traffic congestion. The Armadale Line Upgrade Alliance (ALUA) was formed to deliver this project.

The superstructure of the elevated rail structures consists of two precast L-shaped elements joined by an in-situ slab in lower flange, to form a U-shaped trough girder. To accelerate the construction process, the precast elements are pretensioned and do not require additional post-tensioning. The U-troughs are simply supported on spherical bearings with a guaranteed design life of 120 years to eliminate the requirement for replacing them during the service life of the viaducts. The total height of the L-girders is 2.21 m which is driven by the requirement of the top-flange to act as guardrail to contain rail carts in a derailment case rather than the maximum span of 31.15 m. The viaduct superstructure remains the same throughout. The elevated stations are wrapped around the viaducts and are structurally isolated from the viaducts which has some advantages for passenger comfort while waiting for their services on the platforms.

The piers of the substructure consist of precast elements for columns and headstocks. They are grouped into types, with the emphasis on maximising repetition. Hammerhead piers are used to support the U-trough superstructures between stations, while portal frames are used within the station precent. Station platforms are supported on modified pretensioned girders. The most common precast prestress shape in Western Australia is utilised for those beams.

Deep foundations are required throughout the project, with the majority being continuous flight auger (CFA) drilled shafts piles to optimize construction cycles.

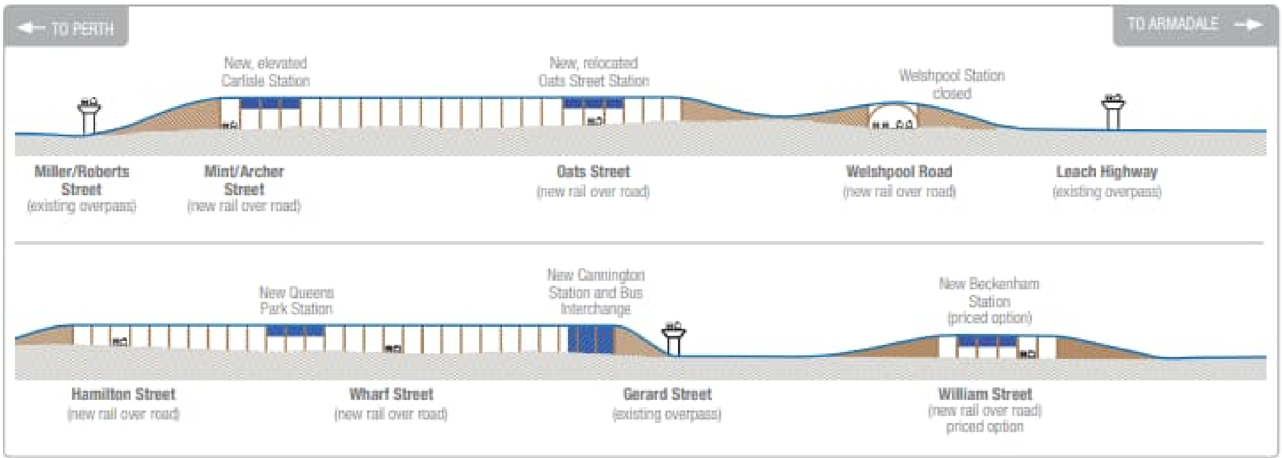

Figure 1. General elevation of the project

2. Design development

2.1 Commonly used L-beams

Pretensioned concrete U-troughs forming viaducts to support rail traffic loads are nothing new. This type of cross section has been used around the world and was recently used at a similar level crossing removal project on the East Coast of Australia in Melbourne. The cross sections developed for the project in Melbourne were therefore reviewed and space proofed for the rail corridor in Perth. There were some differences in the design brief that should be mentioned to further understand the design development. The Melbourne Metro project has standard rail gauge on ballasted track, while the Perth passenger trains run on narrow gauge and the project brief called for track slab design.

The railway corridor for the Armadale line in Perth is generally 24 m wide. Allowance had to be made for two additional tracks in future for the Perth to Bunbury rail line. The cross section developed for Melbourne Metro (refer Figure 2), even after reducing its width for the narrow gauge by approximate 300 mm would have resulted in additional land acquisition for the Perth to Bunbury rail line.

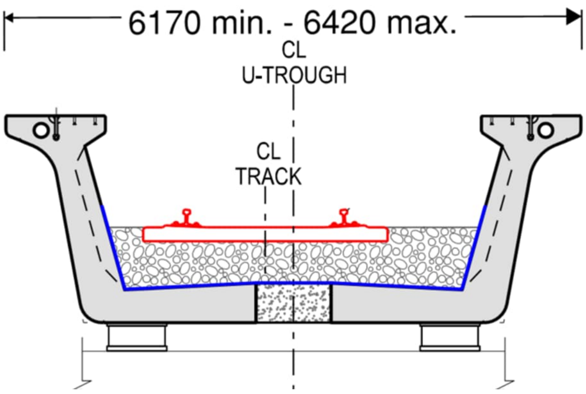

Figure 2. Typical U-trough cross section

2.2 Refinement of the L-beam cross sections

The idea of the Armadale Line Upgrade Alliance (ALUA) was to find a U-trough cross section that can fit four times in the 24 m wide rail corridor to allow easier construction of two additional rail lines in future. To achieve this, a few rail safety, geometric and operational items needed to be resolved to demonstrate that trains will be able to operate safely on the viaduct structures.

Instead of duplicating the cross-section of the Melbourne Metro viaducts, a narrower section was tested. However, reducing the cross section width to approximately 5.5 m during tender design was achieved by having an asymmetric section with minimalist top-flanges on the inside. The idea was to have a combined emergency and maintenance walkway on the outside top-flanges only. In an emergency, the doors of the trains would open on one side only and passengers can evacuate onto the walkway, finding their way to the next station or end of the viaduct. The tender design had two different depth bottom flanges. A 420 mm deep bottom flange for the standard viaduct sections and a 500 mm deep bottom flange for the viaduct sections in the stations. The deeper section in the stations was required to support the higher track slab loads. Due to stations being structurally independent from the viaduct, the station platforms, to provide access to the trains, are cantilevering over the viaduct top flange (refer Figure 4). The top flange in the open viaduct has a secondary function as a containment system in the event of train derailment. In this case, the top flange must be able to take a horizontal load of 500 kN at 1.2 m above track level in accordance with [1]. To allow level access to the trains in the stations, the platform level is 1.085 m above track level. With an allowance for 250 mm deep slab for the station platform and a 20 mm gap to ensure no loads are distributed from the station platform to the viaduct beams. To achieve this the station beam needs to sit approximately 385 mm lower than the viaduct beam. The step was achieved by different height bearing plinth on the first station pier (refer Figure 3).

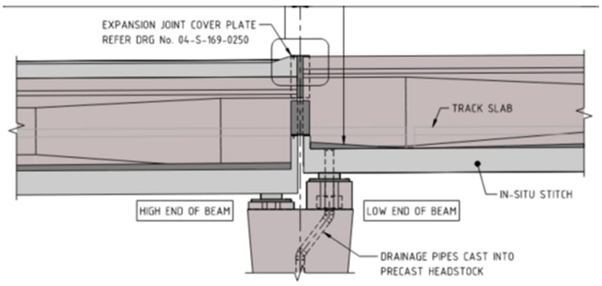

Figure 3. Step from the station beam (left) to the viaduct beam (right)

This concept was the differentiator in the design and won the project. The design team took this concept and simplified the beams to have the same bottom flange depth of 460 mm for both, open viaduct and station viaduct sections. The 460 mm was space proofed to allow for two post tension anchors (VSL Type GC 6-12) to be installed in the bottom flange for each L-beam.

The design team revised and improved further details during the design development after winning the project.

The geometry of the L-beams forming the U-trough were refined. Due to the narrower overall width of the U-trough the bottom flanges of the L-beams are shorter compared to the section of Melbourne Metro.

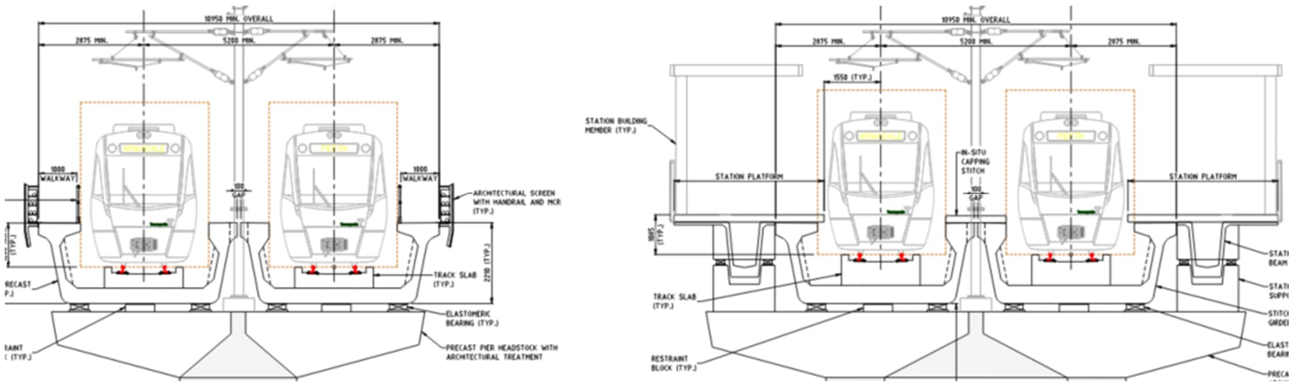

Figure 4. Typical viaduct cross sections. Left: open viaduct. Right: station viaduct

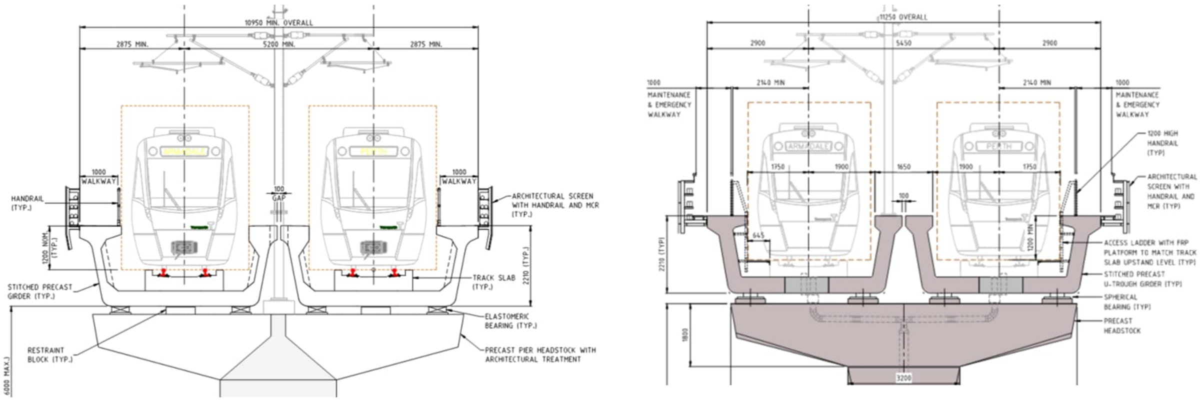

Figure 5. Left: Original design idea at tender. Right: Final design submission for construction

As the Melbourne Metro cross sections were initially used to set-out the U-troughs this resulted in the centre of gravity being outside of the bottom flange L-section causing the precast beams to be unstable when lifted out of the moulds and placed in the storage yard. As storage of the precast beams was essential to have sufficient stock available for rapid deployment. and it needed to be safe, the angle of the webs to the bottom flange were revised from 75 degrees to 80 degrees. This shifted the centre of gravity further to the inside, resulting in stable L-beams during storage – even when stacking two beams they did not need temporary bracing or props to secure them from rotating onto their sides, however stacking three beams propping was required (refer Figure 6).

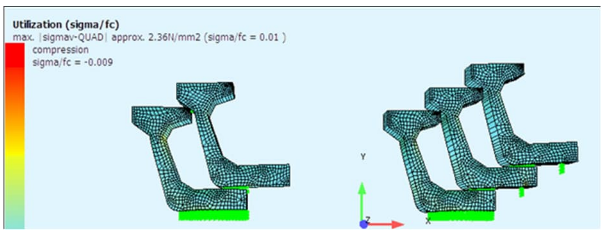

Figure 6. Stacking of L-beam design

Designers allowed for a 1 in 20-year average return interval (ARI) wind load in accordance with [2] to check the stability of the precast beams on the temporary supports in the storage yard.

While revising the angle of the web, the web thickness was also changed to a uniform width to simplify reinforcement scheduling and fixing.

Figure 7. Stacking of L-beams in the stockyard

A minimal cross fall was proposed over the L-beam top flanges to drain the run-off into the U-troughs and collect it there to discharge to the drainage system. The aim was to eliminate water ponding on the top flanges as a safety precaution in a detraining event where passengers are required to evacuate to the next station or end of the viaduct by walking on the outer top-flange. However, analysis demonstrated that the top-flanges will be slightly rotated due to the prestressing forces being asymmetric and the shear centre being off-set from the centre of gravity. As the catchment area is small in comparison with the catchment directly into the viaduct, it was decided to have the top-flanges levelled in the mould. This simplifies the fabrication of moulds and scheduling of reinforcement in the L-beams.

To be able to utilise one stressing bed for the inner and outer beam, the geometry of the top-flanges of the internal and external L-beams was revised to have the same shape on the outside. The inner side the top-flange of the external beam has the same geometry than the internal beam to the first kink and received and 'add-on' to provide the wider top-flange to achieve the minimum clearance requirements for detraining. Ferrules for screens / walkways were included.

The prestressing arrangement and the number of strands at the bottom flange have been optimised to provide an identical strand profile for both external & internal beams (refer Figure 8), to allow construction of internal and external units within one stressing operation. This has significantly reduced the number of stressing operations, improving workmanship.

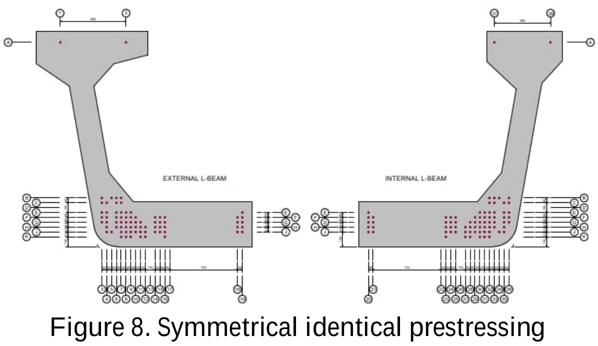

Figure 8. Symmetrical identical prestressing arrangement at the bottom, asymmetric at the top

The prestressing arrangement of the strands at the top flanges is asymmetric with one strand offset by 100 mm for the outer L-beam to balance concrete stresses while maximaising constructability. The 100 mm offset was the maximum the precast manufacturer could allow for to limit the forces this deviation transfers onto the precast strand templates.

A finite element software package with the ability model construction stages of non-symmetric section was used to check the stresses after transfer (refer Figure 9). It was noted that not all established structural software packages will give the correct stress results due to the rotated principal axis.

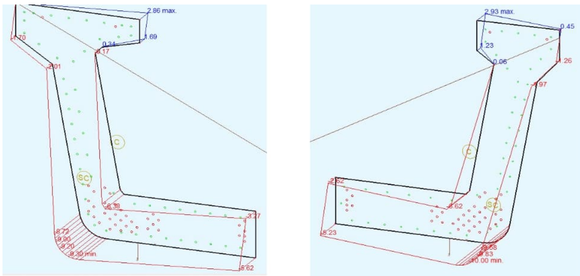

Figure 9. 3D stresses due to prestressing – caused by rotated principal axis

The L-beam stitch joint needed to work for tangent and curved U-troughs. The difference is the clear width between top-flanges of 3.65 m at the tanged sections, widening up to 3.88 m clear width at the curved viaduct sections due to the cant and the rail car geometry. The beams transition from a tangent section to a curved section in one span by providing a slightly wedged stitch pour. The reinforcement needed to be space proved to avoid clashes when allowing for the slight deviation from a perfect parallel stitch joint. Space proofing was done by modelling all beams with 3D reinforcement and tried the assembly of all non-parallel sections virtually. The entire alignment and setting out of span length were reworked to reduce the number of different span lengths to a minimum. At the end, only 5 different length beams were required for the entire project. The adjustment of the different length was generally accomplished by changing the end-diaphragm length of the beams, maintaining the centre typical web section and the web transition section with the same dimension (refer Figure 10). The small variations in length due to the curved alignment were also deducted from the end diaphragm sections. Only for the last 7 spans of the short viaduct section over Welshpool Road, the centre web section length was changed as well, which resulted in an optimised reuse of the steel moulds.

Figure 10. Typical L-beam Type A for length various due to curved alignment

2.4 Interfaces

The precast elements needed to allow for cast in anchors and block outs for walkways, track slabs, the main cable route, the overhead line electrification (OLE), signalling and lighting, drainage, earthing and bonding.

The initial submission had the 1.0 m wide walkway for maintenance and evacuation on top of the concrete top-flange. However, during the further design development, the client insisted on the 2.14 m structural clearance from the track centre to any structural element other than the containment devices (e.g. the top-flanges acting as guardrails). This required a roughly 400 mm widening of the top-flange on the external section. The design team decided to undertake this widening as an integrated walkway-barrier-cable route system to save weight and be flexible with the access requirements around the signalling masts (refer Figure 5 Right).

The idea at tender design to install the OLE masts on the piers was discarded as the distance of about 30 m between masts was too short, specifically where overlaps to end and start the cables was required. The OLE masts were installed on one of the inner flanges of the viaduct, which required exact allocation of the cast-in anchor needed to be coordinated and tabulated in the design drawings.

The main cable route and signalling cables needed to be space proofed and a system developed to install them in the walkway barrier. Where signalling masts were required, they needed to be safely maintainable, which resulted in wider walkways to allow safe passage in an emergency and sufficient space to have safe access to replace lights (refer Figure 11). The reinforcement to allow for shear transmission from the track slab to the viaduct structure was designed to be integrated in the in-situ concrete stitch to avoid any further cast-in items to the L-beams. The block outs for the earthing and bonding cables were cast into the top-flanges near the locations of the signalling masts.

The internal handrail length was reviewed to allow the best possible free detraining access in the case a train has to stop on the viaduct outside of a station and passengers need to leave the train.

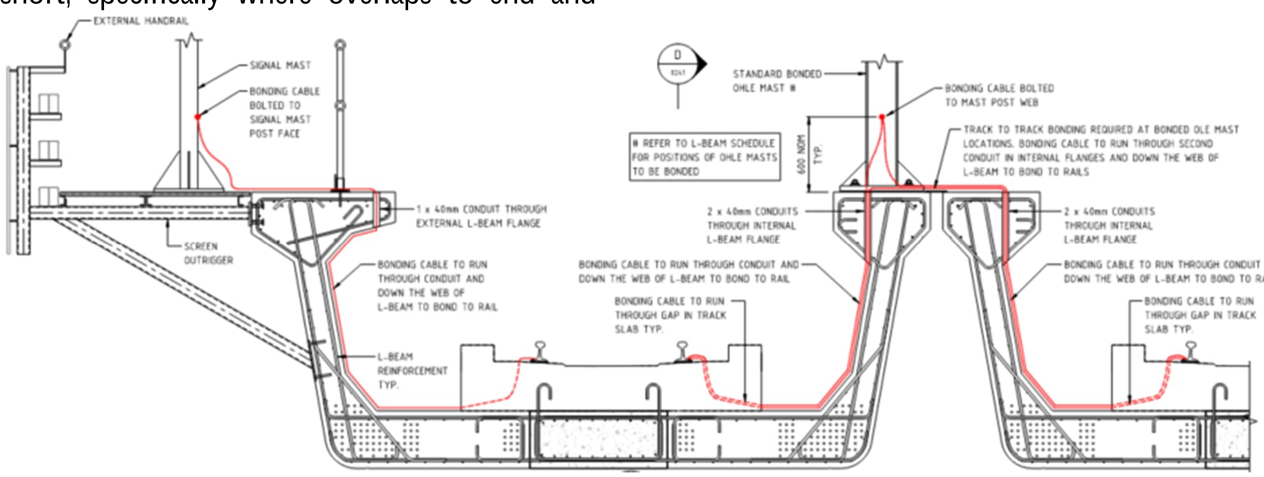

Figure 11. Track slab reinforcement, walkway widen at signalling mast and earthing and bonding

plunged into blocks outs and grouted after the beams have been aligned. The temporary support was provided by an elevated grout pad installed on the headstock aligning with the centre of the bearing base plate. A typical bearing layout for a span can be seen in Figure 13.

The drainage system was relocated from the precast section to the in-situ concrete stitch and the track slab was designed to allow for free water flow to the scuppers discharging into pipe collectors installed in the precast headstock elements (refer Figure 12).

The simplifications resulted in turnaround of two beams per day for each mould after the crew in the precast yard has their routine.

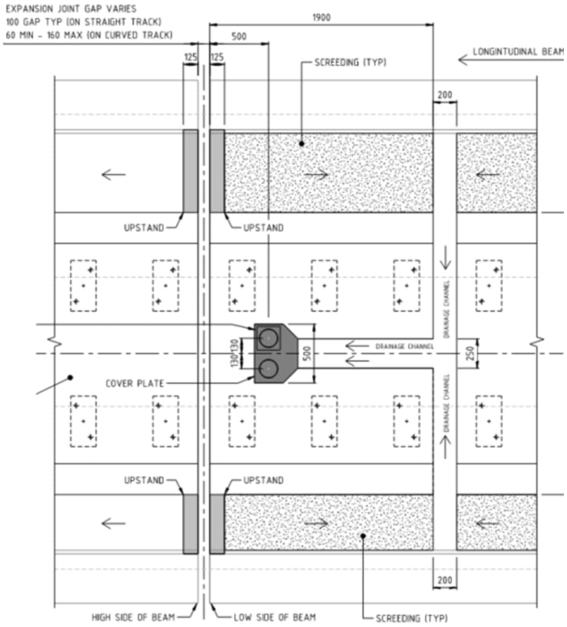

Figure 12. Drainage integrated around track slab

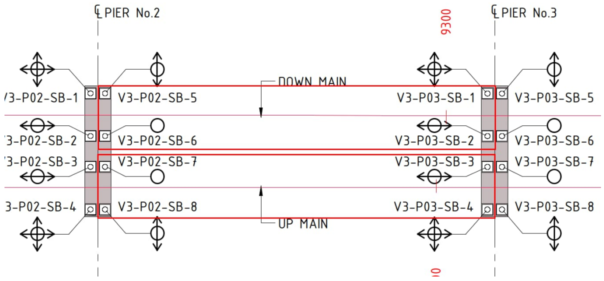

Figure 13. Typical bearing layout

2.5 Decision on bearings

The tender design was based on elastomeric bearings. The shear forces to resist the nosing and seismic loads in the transverse direction and the braking and traction loads in the longitudinal direction were too high for the elastomeric bearings to resist them. Shear keys were required to provide sufficient capacity to resist those loads. The design team suggested to use spherical bearings as an alternative and tested if the difference in detailing provided any benefits. The bearings were designed and detailed to allow for the longitudinal grades of the viaduct, which eliminated the requirement to install tapered steel plates under the viaduct superstructure. The spherical bearings were installed under the L-beams prior to lifting them onto the headstocks. The shear studs of the bearings were

Revising the bearings from elastomeric to spherical type was also endorsed by the asset owner as the spherical bearing manufacturer provided a certificate for a 120-year design life. The elimination of the shear keys was not the only benefit the change to spherical bearings provided. Spherical bearings are more compact, and the smaller bearing footprint resulted in a reduction of headstock width by 700 mm. The savings achieved in concrete quantities on the 132 headstocks and 532 L-beams. It should be mentioned that the spherical bearings needed to allow for isolation to electrically isolate the superstructure from the substructure in the stations.

2.6 Precast pier columns and headstocks

An 18-months shut down period meant the viaduct structural works needed to be completed within 7 months to allow for installation of the track slabs, the main cable route, the overhead line electrification, signalling and lighting, drainage, earthing and bonding and the station structures. To be able to construct the four kilometres of elevated viaducts within the 7 months the Alliance had precast as many components as possible.

The foundations needed to be piled foundations due to the ground conditions. To accelerate construction, the geotechnical team, together with the site team, designed and installed multiple mock-up pile set-out for in-situ pile testing of CFA pile to get an approval in principle by the client. CFA piles are usually not allowed for bridge structures in Western Australia. However, due to their higher geotechnical capacity when compared to bored piles and quicker installation methodology, it was the preferred pile option for the Alliance. Four CFA piles with a diameter of 1050 mm and 1200 mm were required for each pile cap supporting a pier column.

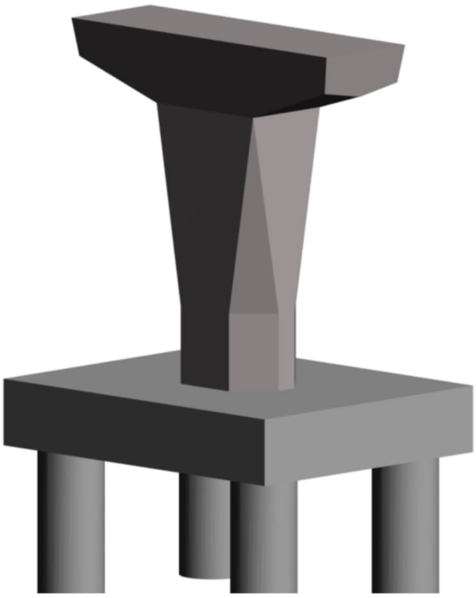

Figure 14. Typical 3D shape of the pier columns and headstocks

The pile caps were cast in-situ to allow for the tolerance of pile installation and to be flexible to add a fifth pile in centre should geotechnical capacity not be achieved due to changing ground conditions along the alignment. Precast elements were developed to start above the pile cap. The architecturally driven pier shape required multiple precast elements for the substructure to utilise the moulds as many times as possible without having to adjust them. The piers were consequently split into a headstock, the upper precast column element with the taper and the lower precast column element with the hexagonal shape (refer Figure 14 and Figure 15).

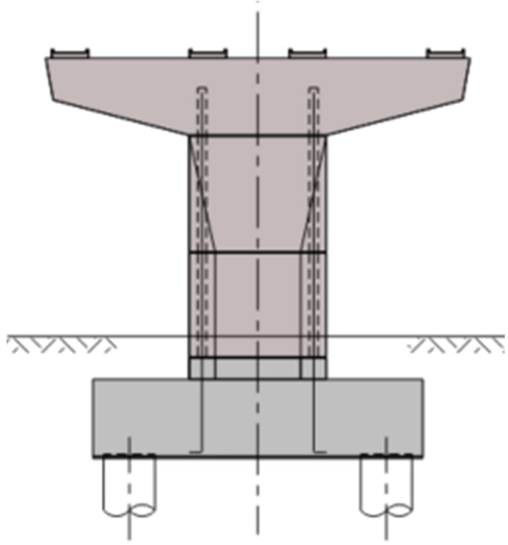

Figure 15. Elevation of a typical pier with the different elements

2.6.1 Headstocks

The headstocks needed to utilise the same mould for tangent and curved alignments although on the curved alignment the headstocks are 250 mm wider. They had to allow for the bearing plinth, the temporary tie-down to secure the L-beams and to receive the drainage scuppers from the superstructure. The structural connection to the piers and the pile caps was achieved by allowing for block-outs to place the headstocks over the anchor bars that extend for the full length from the pile cap. To align the anchor bars in the pile cap to precisely fit into the pier column elements and the headstock elements, steel templates were used (refer Figure 16). The block-outs were injected with grout to generate the bond between the precast concrete elements and the anchor bars from the pile caps.

Figure 16. Anchor bars with templates installed in pile cap

2.6.2 Pier columns

Pier elements were dived into three segments. The upper part is the same for all piers. It includes the transition from the hexagonal cross-section to the rectangular cross-section. The height for this element was based on the height requirement of the shortest pier in the projects.

The lower part is a hexagonal section with a minimum height of 1.0 m and a maximum height of 2.9 m. Height intervals of 100 mm were used in combination with the in-situ plinth on top of the pile cap to achieve the exact headstock top-level (refer Figure 17).

Figure 17. Pier columns with lower and upper elements for a station headstock

2.6.3 Reinforcement

The reinforcement of all precast elements was modelled in 3D. This allowed all reinforcement cages to be pre-manufactured (refer Figure 19) and lifted into the moulds. To avoid deflection and distortion of the reinforcement, cages templates made from steel plate were used. Those templates were cast in the precast elements and were essentially lost. However, it allowed an accelerated construction if the precast elements and was well worth it.

Figure 18. Gantry cranes for L-beam installation

3. Installation of the viaduct

For the erection of the viaduct elements, a mixture of mobile cranes and gantry cranes were used. The Alliance purchased four purposely made self-tracking gantry cranes (refer Figure 18) which were utilised to transport beams in the stockyard and later to lift the beams of the self-propelled modular transporter (SPMT) units onto the headstocks. The gantry cranes were spanning over the two side by side U-troughs, which allowed up to 8 beams to be installed each shift. The pier column elements were lifted into position using mobile cranes.

Figure 19. Pre-manufactured reinforcement cages for headstocks and viaducts

4. Summary

The construction of the Victoria Park - Canning Level Crossing Removal Project is well on the way. The milestone for completing the viaduct construction has been met with completing the L-beam installation on 20th of July 2024 after possession of the tracks was given on the 23rd of November 2023. The design effort of simplifying the entire viaduct system to reduce the number of elements to a minimum has worked well for construction. The installation methodologies for the precast elements required minimal adjustments.

The optimisation of the L-beams to eliminate the post-tension ducts has saved a large amount of time on site. While undertaking the design of L-shape beams which are joined to U-troughs, designers need to be aware of limitations of the software they chose. Not all established design software packages can accumulate the stresses correctly.

Figure 20. Completed viaduct

5. Acknowledgements

The Armadale Line Upgrade Alliance (ALUA), consisting of Acciona, BMD and WSP, won the competitive tender to deliver this project under an Alliance Agreement with the Public Transport Authority (PTA) Western Australia. The project is jointly funded by the Australian and Western Australian Governments to future proof the METRONET rail network and improve congestion on the road network.

Figure 21. Viaduct partially completed

References

[1] AS 5100 Part 2: Scope and general principles, Standards Australia Limited, 2017.

[2] AS/NZS 1170 Structural design actions Part 2: Wind actions, Standards Australia Limited, New Zealand Standards, 2021.