Ashton Avenue Integral Bridge

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ABSTRACT

The Ashton Avenue Bridge is a replacement of the original three-span timber bridge over Perth-Fremantle Rail line in Claremont Western Australia. The replacement bridge comprises a single span of 19.1 m with a trafficable width of 11 m between kerbs, and a 3.5 m shared path on each side of the bridge.

There were a number of constraints that drove the design development. The first key constraint was that due to the road vertical alignment and required rail clearances, the span to depth of the bridge was limited to minimum 37 at mid-span. The second key constraint was that, due to the bridge spanning operating rail, construction was restricted to one weekend shutdown for installation of the deck planks, while the rest of construction activities were to be completed during normal rail operation.

In order to accommodate these constraints an innovation solution was developed, a combination of a superstructure consisting of 20 No precast battledeck planks with a cast in-situ reinforced concrete deck slab and integral abutments supported on piled foundations. The integral abutments allowed the battledeck planks to act compositely in both sagging and hogging, reducing the maximum sag moment and allowing a superstructure depth of 515 mm.

This paper will outline the constraints and the design requirements that characterised the design of the battledeck planks and the integral abutments, and then will discuss the key features in the design and construction of the Ashton Avenue Bridge.

Keywords, Battledeck plank, integral bridge.

1 INTRODUCTION

The bridge site is located in Claremont, to the west of Perth CBD, adjacent to the intersection of Ashton Avenue and Gugeri Street, and passes over Perth-Fremantle rail line (See Figure 1). The original Bridge No. 903 was constructed in 1914 and consisted of 3 spans totalling 25.4 m in length, carrying two lanes of traffic and a footpath, (refer Figure 2). Due to deterioration of the main timber members, the bridge had been posted with a gross vehicle load limit of 12 t, which restricted vehicle movement such as buses. In addition, ongoing deterioration and deficiencies had caused significant maintenance costs; hence, it was decided that replacing the bridge was the only viable option. The replacement bridge was required to be designed to carry SM1600 / HLP400 loading in accordance with AS 5100: 2004 and be installed at the same location of the old bridge.

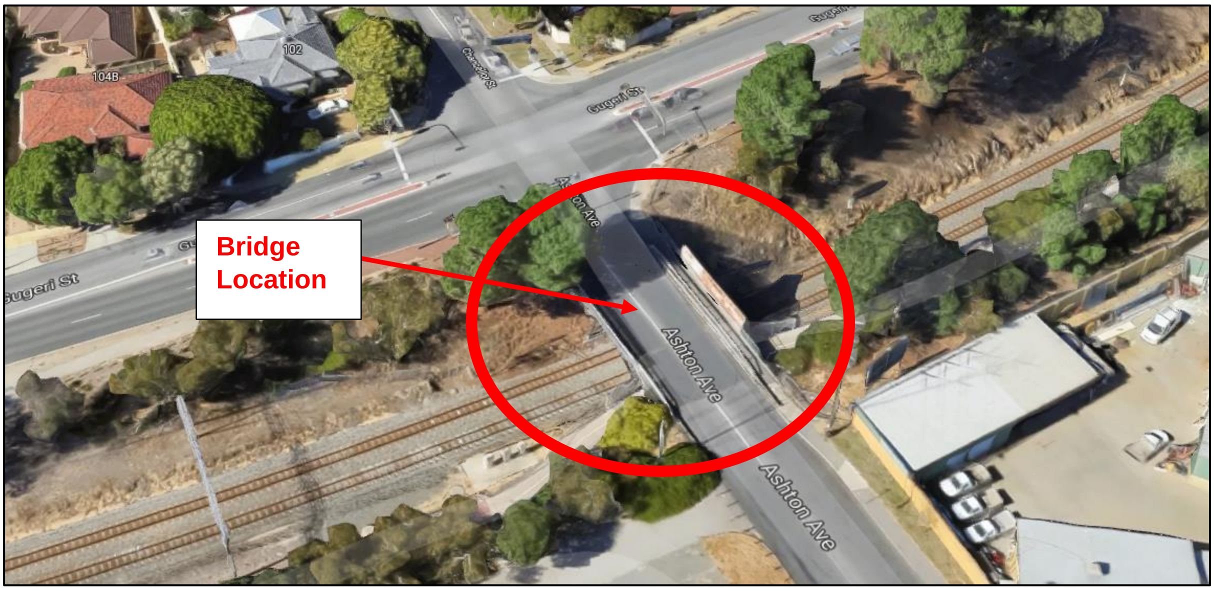

Figure 1 Bridge Location

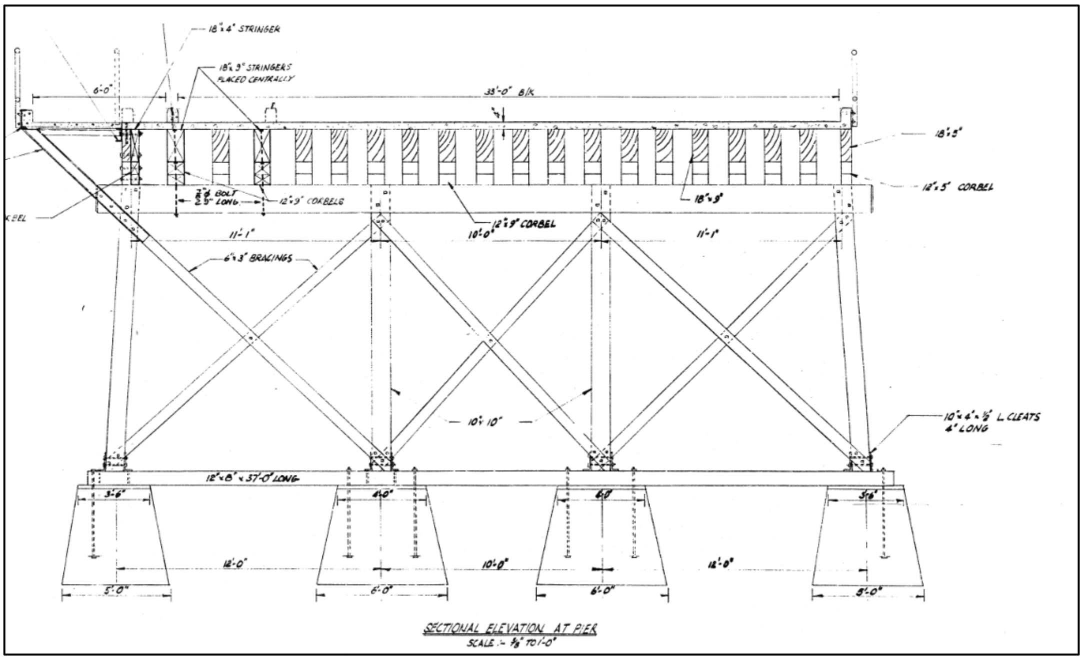

Figure 2 Timber Bridge built in 1914

1.1 Replacement Bridge

In developing the appropriate bridge replacement option, there were a number of design constraints that had to be accommodated:

- The Ashton Ave Bridge alignment was fixed by the alignment of old bridge.
- The replacement bridge was to be a single-span structure with 2% one-way crossfall and a span length of 19.06 m;
- The minimum vertical clearance required over the rail lines to be 5.4 m (plus an additional 60 mm for settlement, deflections and tolerances) which was an increase clearance of 600 mm compared to the original bridge;
- Constant deck thickness to provide minimum clearance over shared path;

- Very limited space for foundation works due to constrained corridor;
- Services including:
- PTA services within rail corridor;
- Water Corporation water pipe within bridge;
- Western Power power lines, including large; transmission pole at SW abutment;
- Only two weekend rail shutdowns permitted – one for deconstruction of the existing bridge and one for planks placement. All other work to be completed during normal rail operations; and
- Due to the proximity of the bridge to Ashton Avenue intersection with Gugeri Street, there was limited opportunity to increase the road level.

In order to develop an appropriate structural form and solution for this bridge, the key design consideration was to provide a design that allowed the required slender superstructure depth. Based on the above constraints, the maximum permitted superstructure depth was 515 mm (span-to-depth ratio of 37).

1.2 Replacement Options

Three options were considered in developing the concept design, they comprised:

- Option 1. The VFT-WIB beam;
- Option 2. Battledeck Planks; and
- Option 3. Prestressed concrete planks with an in-situ topping slab.

All three options were designed with integral, portal frame type abutments to minimise the structural depth. Evaluation of these options determined that the battledeck plank was the most appropriate solution. Some of the advantages of battledeck planks are:

- Battledeck acts as a composite beam on both sagging and hogging part and provides a constant deck thickness, allowing PTA to have their required clearance under the whole bridge;
- The difference between the long-term and short-term deflection of battledeck plank is minimum compared to the conventional prestressed reinforced concrete beams;
- It has a simpler steel fabrication compare to VFT-WIB. The steel beams do not require the detailed fabrication which the VFT beams require for their laser cut continuous composite dowel profile along the webs and hunched beam profile.

1.3 Selected Option

The proposed superstructure comprises of 20 No. 330 mm deep precast battledeck planks, acting compositely with a 185 mm thick (minimum) in-situ concrete deck slab, giving a total depth of superstructure of 515 mm. Each plank comprises 2 No. 310 UC 158 steel beams, cast into concrete to form precast planks.

To maintain clearance of 5.4 m from the rail line, a restricted span to depth ratio of 37 was required. To achieve this stringent limitation, an end fixity was provided to reduce the section size by transferring sagging bending moment to hogging area.

The steel flanges act as the main longitudinal reinforcement in sag, while the hog capacity is resisted by the combined action of the steel beam and reinforcement in the topping slab. A combination of shear studs and reinforcement provides composite action with the topping slab. The battledeck planks were cast into the abutments and connected to the abutments by Macalloy anchor bolts. The upper part of abutments and the topping slab were cast together to provide sufficient end fixity for the planks.

The substructure comprised reinforced concrete integral abutments supported on a piled footing with two rows of 450 mm bored piles. Figure 3 presents the general arrangement of the replacement bridge.

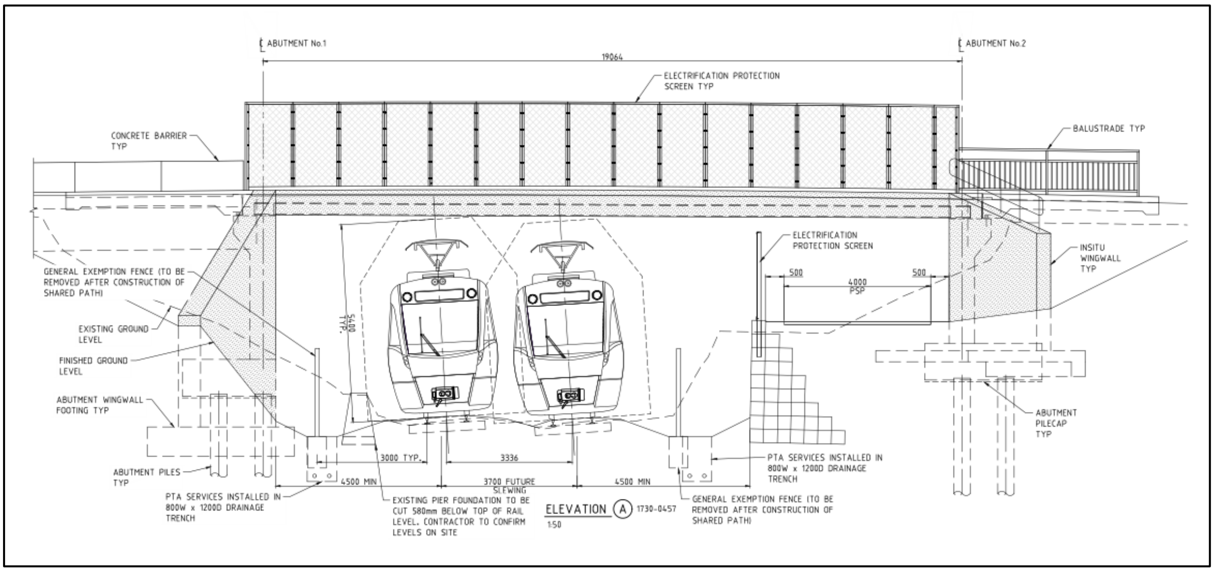

Figure 3 Typical Longitudinal Section

2 ASHTON AVENUE INTEGRAL BRIDGE DESIGN CONSIDERATIONS

2.1 Superstructure

2.1.1 Battledeck plank geometry

The composite steel beams encased in concrete were designed in accordance with the requirements of AS5100.6 for composite compression members and EN 1994.2 for filler beams. According to EN1994.2, filler beams are defined as partially concrete-encased rolled or welded steel beams encased in in-situ concrete and having their bottom flange at the level of slab soffit. The code requirements for filler beam were found to be applicable to battledeck planks and were considered in superstructure design.

To determine the geometry of the plank the following design requirements were considered [10]:

- To consider the bond between the concrete and steel, the concrete cover to the side of an encased steel flange shall be minimum 80 mm [10]. The minimum cover is mentioned as 40 mm for compression members in AS5100.6-2017, Cl 10.6.1.3;
- The clear distance S_f between the upper flanges of the steel beams shall not be less than 150 mm, so as to allow placement and compaction of concrete;
- The minimum longitudinal and transversal reinforcement are discussed in Section 2.1.2 & Section 2.1.3;
- Normal-density concrete shall be used.

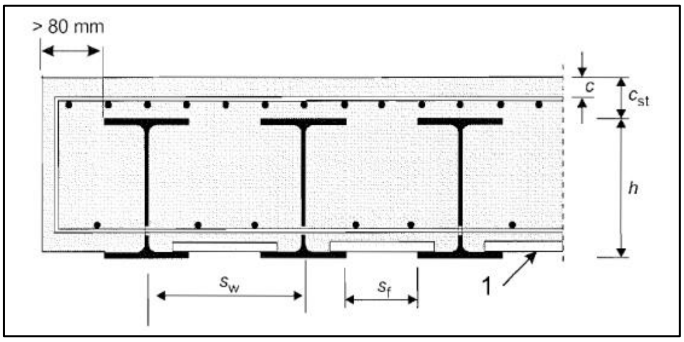

Figure 4 Cross section of composite section encased in concrete [11]

2.1.2 Battledeck Minimum Longitudinal Reinforcement

To provide sufficient ductility for the beams a minimum area of reinforcement within the effective width of the concrete flange should be provided to satisfy the following condition [11].

$$A_s \geq \rho_s \cdot A_c$$

$$\rho_s = \delta \cdot \frac{f_y}{235} \cdot \frac{f_{ctm}}{f_{sk}} \cdot \sqrt{k_c}$$

Where:

- A_c is the effective area of the concrete flange;
- f_y is the nominal value of the yield strength of the structural steel in MPa;
- f_{sk} is the characteristic yield strength of the reinforcement;
- f_{ctm} is the mean tensile strength of the concrete and is defined as 0.3.f'c2/3;
- k_c is a coefficient which takes account of the stress distribution within the section immediately prior to cracking, and can be considered conservatively as 1;
- δ is equal to 1.0 for non-compact cross-sections, and equal to 1.1 for compact cross-sections at which plastic hinge rotation is required.

Based on the above formula the minimum longitudinal reinforcement for battledeck plank was determined as 1.1%.

The clear distance between longitudinal reinforcing bars and the structural steel section should be smaller than required as set out in AS 5100.5, even zero. However, in this project a minimum clearance of 25 mm was adopted. Additionally, the aggregate size was limited to 14 mm to minimise the risk of voids between the steel sections and reinforcement.

2.1.3 Battledeck Shear Reinforcement.

According to the conducted laboratory test by ASCE [13], the battledeck planks will act as an integral unit under the live load action. However, a special reinforcement arrangement would be required for the precast planks to ensure that the load is distributing evenly to both steel beams especially in the construction stage, before casting the topping slab.

Based EN 1994.1.2 [9], for a web of a concrete-encased section, the concrete that encases it should be reinforced and mechanically connected to the steel section. It should also be capable of preventing buckling of the web and any part of the compression flange towards the web. To meet the above requirements the following reinforcement arrangements were investigated:

- Welding shear reinforcement to the web of steel section (Figure 5-a.),
- Passing the shear reinforcement through the web of the steel section. (Figure-b); and
- Welding the studs with a diameter greater than 10 mm welded to the web (Figure 5-c).

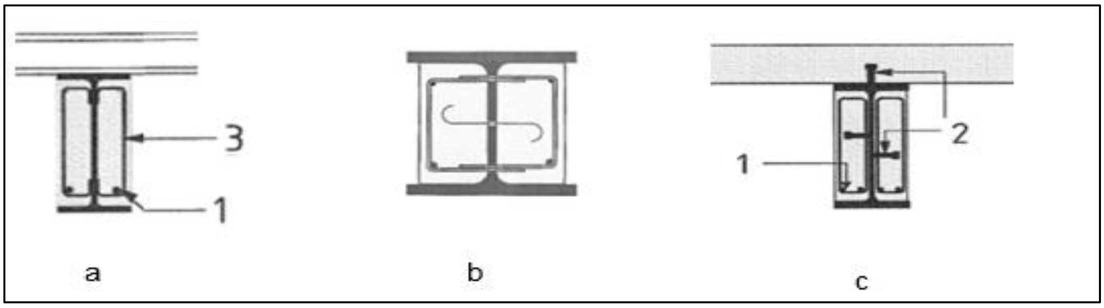

Figure 5 Arrangement of Stirrups [11]

To investigate the constructability of the proposed detail, some fabricators were approached and the first option was determined as the most economical and practical option. Figure 6 shows the final arrangement of stirrups and shear studs in each battledeck plank of Ashton Ave Bridge.

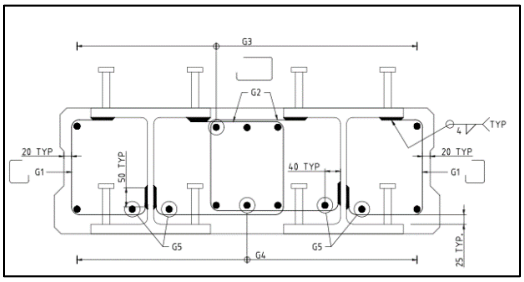

Figure 6 Arrangement of stirrups in the superstructure

2.1.4 Sag Bending Resistance

The ultimate bending resistance moment of the battledeck planks was determined by plastic theory (refer Figure 7), assuming the beams were laterally supported during the construction. The accuracy of this assumption was proofed by calculating the beam's unbraced length and making observations during construction. The full shear connection was provided between the structural steel section and the web encasement in accordance with EN 1994.2 Cl 6.6 and AS 5100.6 Cl 6.6. The axial force due to soil pressured and ambient temperature was considered in reducing the bending capacity

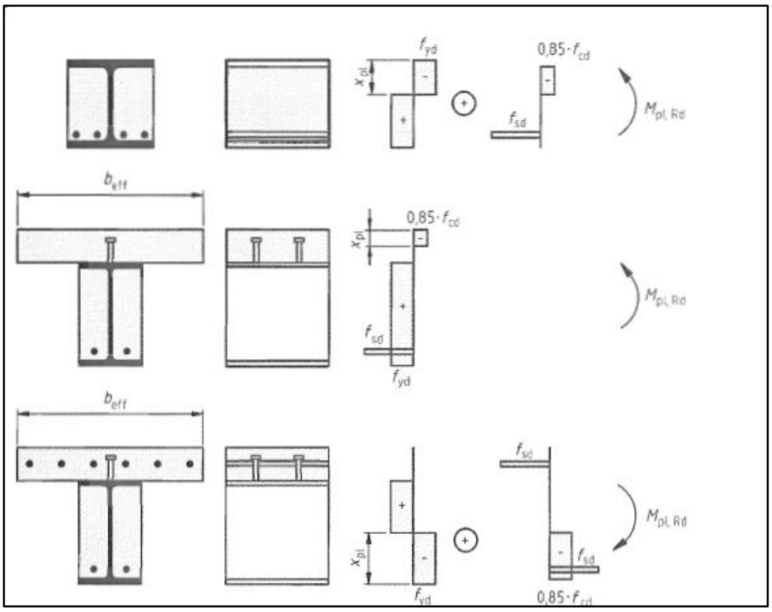

Figure 7 –Plastic stress distributions for effective sections [11]

2.1.5 Shear Resistance

EN 1994.1.1, indicates that the resistance to vertical shear should be taken as the resistance of the structural steel section. The contribution of the web encasement to shear resistance may be taken into account for the determination of the design shear resistance if stirrups are used in accordance with Figure 5 and they are attached to the web by full strength welds, otherwise the contribution of the shear reinforcement should be neglected. In this project, the contribution of concrete on shear capacity was ignored and the steel beams were conservatively designed to carry the shear force. This was partly because the concrete at the end of the planks was cast simultaneously with topping slab; therefore, the shear force due to plank self-weight and topping slab were applied only to the steel beams at the plank ends (refer Figure 8). Additionally, the bending capacity was reduced where the design vertical shear force exceeds half of the design plastic resistance of the structural steel section.

Figure 8 –Precast battledeck plank

2.1.6 Providing End Fixity.

Providing the end fixity to transfer bending moment from beam to abutment was one of the challenging parts of this project. Three options were investigated for transferring the bending moment from beam to abutment.

Option 1: Using Shear connectors on steel beam flanges

In this option, the bending moment is transferred by installing shear connectors on both top and bottom flanges of steel beams, as shown in Figure 9. Due to space limitations, hoop shear connectors were considered to be more effective than the usual stud connectors. However, hoop connectors are much more expensive than stud connectors [7].

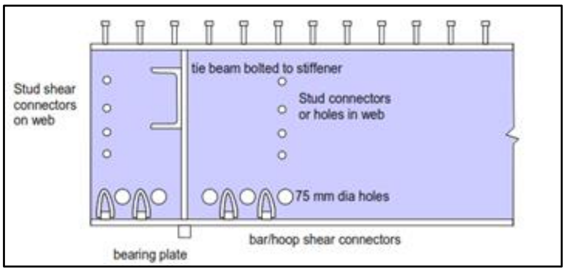

Figure 9 –Using shear studs to transferring bending moment from steel girder to end abutment. [7]

The end bending moment imposed a horizontal shear force. As the planks were very shallow and the horizontal force was large force, providing the necessary shear ligatures in the limited space was considered to be difficult.

To assess the constructability of this detail and the connection joint, a 3D model was created. This allowed the reinforcement layout to be assessed. Based on this model, it was determined that due to congestion of reinforcement, it was likely that concrete would be unable to flow around the reinforcement adequately and that voids may form. Hence, it was considered that this option was not feasible.

Option 2: Using Endplate to provide end fixity.

In this option, the shear connection between the main girders and the abutment is provided by means of an endplate to the girder, with shear stud connectors, as shown in Figure 10.

This option had similar issues as Option 1, in regard to limited space and congestion. Furthermore, due to the size of the end plate, installing shear ligatures or ties was more difficult than Option 1.

Figure 10 –End Plate Connection. [7]

Option 3: Using Macalloy Anchor bolts.

This option utilises a combination of shear connectors and Macalloy anchor bolts to transfer the bending moment. The steel beams were cast into in the abutment and connected to the abutments by Macalloy anchor bolts. The anchor bolts moved the stress concentration away from the connection joint, hence allowing sufficient room to provide required shear ligatures and longitudinal reinforcement. This option was selected as the preferred option and was constructed successfully. Figure 11 shows the end connection detail.

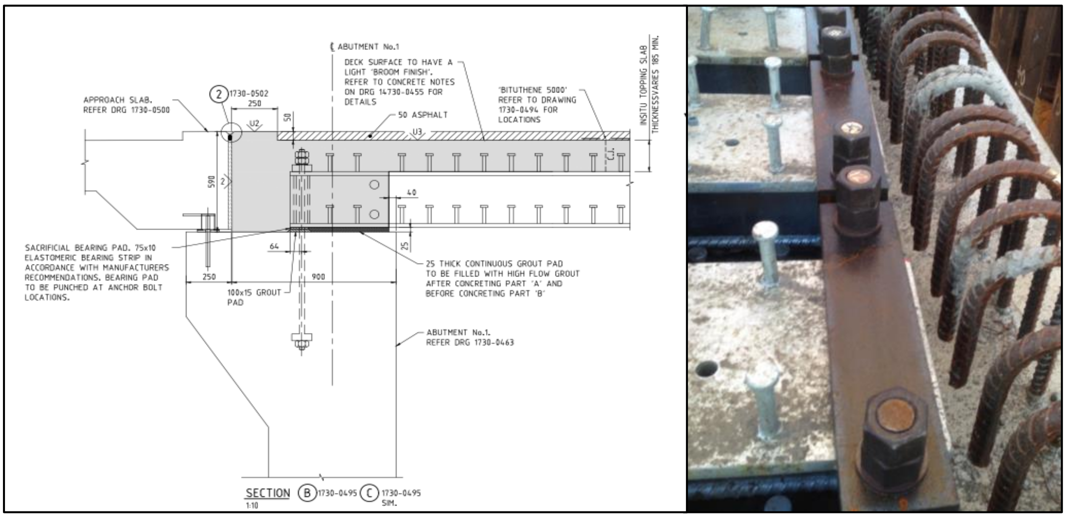

Figure 11 –Using Macalloy anchor bolts

2.1.7 Pre-camber

The deflection of planks during construction had to account for the variation in loads as well as the change to constraint conditions, including the transition from a determinate arrangement to an indeterminate arrangement. The drawings presented the required pre-camber to negate the final deflected shape in order to achieve the final deflected shape project. The beams' remaining hog was monitored during construction, and the construction staging was adjusted to prevent excessive deflection based on the recorded plank hogs

2.1.8 Superstructure Construction Staging.

The abutment was built in two principal stages, the first stage up to the soffit of the slab and on the second stage around the deck girders, providing an integral connection of the deck to the abutment. To aid lining and levelling of the main girders, a sacrificial rubber bearing was set into the lower part of the abutment. This allowed the planks deflect and rotate freely during the construction.

The design was based on the deck slab being poured in two construction stages. In the first stage of the deck pour, a 15 m long strip at the midspan of the deck was to be poured. In the second stage, the remaining part of the slab was to be poured. The aim of this construction sequence was to minimise the hogging bending moment and allowing the planks to rotate freely under their self-weight and the weight of the fresh concrete.

This planned staging was modified during construction. After placing the planks, it was noticed that two of the planks had about 20 mm less remaining hog than the other planks. The reason was unknown, as all the beams had a similar reinforcement arrangement and detailing. Based on the actual remaining hog, the final deflection of the planks was calculated and it was noticed that the above mentioned planks would have sag deflections under the permanent load which was not acceptable. To rectify the issue, the first option was rejecting the planks and fabricating new ones. However, this would cause a significant delay on the project and would have a substantial impact on the local community. Hence, to solve the issue and to reduce the deflection of the beams under permanent loads, another construction stage was added. Based on the revised construction staging, a 10 m long strip at the midspan was poured. The second stage involved pouring a 1.5 m strip adjacent to each abutment. In the final, third stage, the remaining part of the slab was constructed. The adjustment in the construction staging had the following impacts on the superstructure:

1- By reducing the area of the first pour, the planks deflection was reduced;

2- During the third stage of deck pour, the concrete at midspan had reached its final strength. Consequently, the planks at the midspan were acting compositely with the topping slab, and their deflections under fresh concrete weight were much less compared to the non-composite sections. In addition, providing the end fixity minimised the plank deflections further; and

3- Due to this construction staging, the hogging bending moment increased slightly. The superstructure and the substructure were checked for the additional hogging bending moment and were found to be satisfactory.

2.1.9 Shrinkage and Gradient Temperature effect

The effects of shrinkage and differential temperature were only considered in the serviceability limit state and were neglected in the analysis for the ultimate limit state. This is in-line with AS 5100.6 & EN 1994.1.1. Also, for cracked sections, the primary effects of shrinkage were neglected when verifying stresses for the secondary effects.

2.2 Integral abutment

Integral abutment or jointless type bridge structures are single or multiple span continuous structures that have their superstructure cast integrally with their substructure and they accommodate superstructure movements without conventional expansion joints. Currently, about 87% of the bridges less than 100 m long in the United States have integral abutments. The advantages of integral abutments are:

- Moment redistribution: The integral abutments provide end fixity, which will cause moment redistribution from sagging to hogging bending moment. Figure 12 shows the bending moment redistribution in the superstructure planks of Ashton Ave Bridge under live load. By using integral abutments, 329 kN.m of sagging bending moment under live load was redistributed toward hogging bending moment

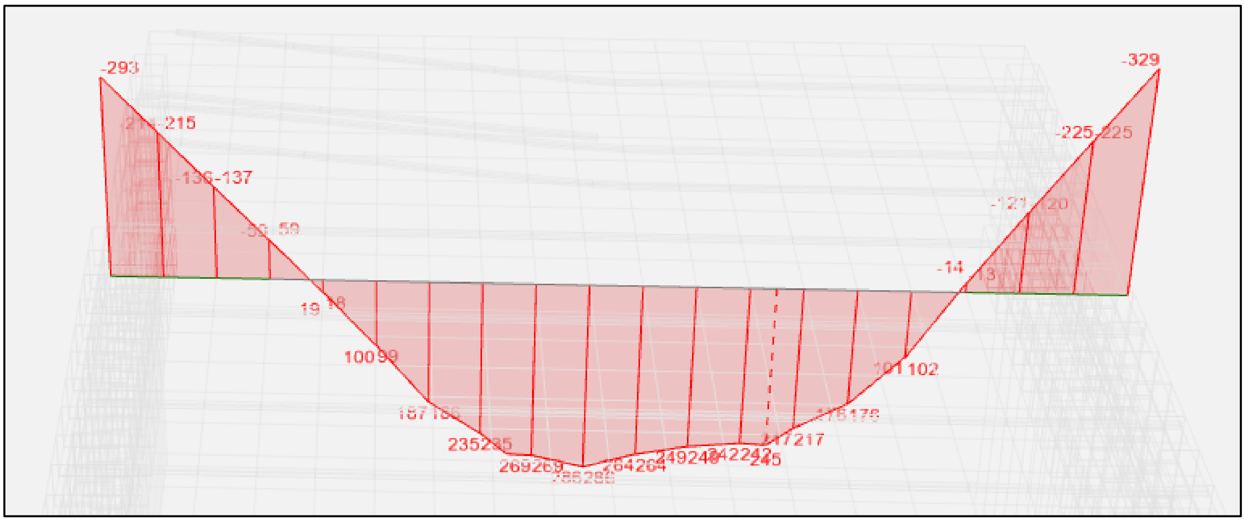

Figure 12 –Plank bending moment diagram under live load

- Operations and maintenance advantages including:
  - Improved long-term serviceability;
  - Improved ride-quality and noise reduction;
  - Reduced maintenance requirements leading to the elimination of the hazards associated with bearing and joint inspection and their maintenance;
  - Lower whole-of-life cost; and
  - Improvement of bridge appearance through the elimination of staining caused by water leakage through joints.

The following sections outline the key criteria and considerations for designing integral abutments.

2.2.1 Geometry requirements,

Neither Eurocode nor Australian code covers the design of integral bridges. Vicroads BTN 010 [4] has a brief general design guideline for integral bridges geometry requirements. PD6694 [6] provides comprehensive guidance to make a realistic estimate of the pressures that will develop with time behind integral abutments.

The following geometry criteria were considered while selecting the integral abutment for Ashton Avenue Bridge:

- The maximum overall length is limited to 70m; this limit is aimed to controlling the maximum passive soil pressure arising from thermally-induced cyclical displacements at the abutments, which will, in-turn, limit the maximum stresses in the piles and the magnitude of cracks between the approach slab and approach pavement [6];
- The maximum skew angle for integral abutment bridge designs shall be 30 degrees [3]&[5];
- Superstructure configurations that require the use of horizontally curved girder schemes shall also preclude the use of integral abutment bridge designs [1];
- BA 42/96 [3] recommends that the integral bridge shall not be used where the induced cyclic movement of each abutment exceeds ±20 mm. However, the more recent guideline PD6694 limits the maximum movement to 40 mm.

2.2.2 Strain ratcheting [6]

Integral bridges accommodate expansion and contraction of the superstructure through movement of the abutments in and out of the backfill. In granular soils, this repeated contraction and expansion (generally after 120 thermal cycles) causes particle realignment and an associated increase in the soil stiffness and the mobilized passive resistance. This effect results in a progressive year-on-year increase in soil pressure which is termed "soil ratcheting".

The enhanced earth pressure caused by soil ratchetting (K^*) is dependent on the total movement of the end of the deck from its maximum contraction position to its maximum expansion position. The characteristic value of the movement (d_k) *is given by* $d_k = \alpha L_x (T_{e;max} - T_{e;min})$

Where:

- α is the coefficient of thermal expansion of the deck;
- L_x is the expansion length measured from the end of the bridge to the position on the deck that remains stationary when the bridge expands;
- $T_{e;max}$ and $T_{e;min}$ are the characteristic maximum and minimum uniform bridge temperature components for a 50-year return period

The design value of d_d may be found from the following equation:

$d_d = ½ . d_k (1 + \gamma_q . \psi)$

The γ_q & ψ shall be determined using EN1990 codes. For designing Ashton Ave Bridge, the values of 1.55 and 0.6 were considered for the above factors.

For a piled or embedded integral abutment with cohesive soils, the effects of strain ratcheting in the cohesive soils may be ignored, and the pressure on the wall and piles when the end of the deck expands are calculated using a conventional soil–structure interaction analysis

2.2.3 Horizontal soil pressure on integral abutments

The horizontal soil pressure on integral abutments is dependent on the type of the foundation and the method by which the thermal movement would be accommodated.

2.2.3.1 Horizontal pressures on abutments accommodating thermal movements by rotation and/or flexure [6]

According to PD 6694, for integral abutments which accommodate thermal movements by rotation and/or flexure, such as full height abutments on spread footings, the design value of the earth pressure coefficient for expansion K* may be calculated from the equation shown below, but should not be taken as greater than Kp.

$$K^* = K_0 + \left(\frac{C\, d'_d}{H}\right)^{0.6} K_p$$

where**:**

- H is the vertical distance from ground level to the level at which the abutment is assumed to rotate; that is, the underside of the base slab for rotationally flexible foundations and the top of the base slab for rotationally rigid foundations. For an abutment wall that is pinned at or near its base, H is measured from ground level to the level of the pin;
- d'_d is the wall deflection at a depth $H/2$ below ground level when the end of the deck expands a distance d_d may conservatively be taken as 0,5d for abutments with rotationally rigid foundations and 0,7d_d for abutments with pinned walls rotationally flexible foundations;

C is 20 for foundations on flexible (unconfined) soils with E<100 MPa; and is 66 for foundations on rock or soils with E> 1000 MPa;

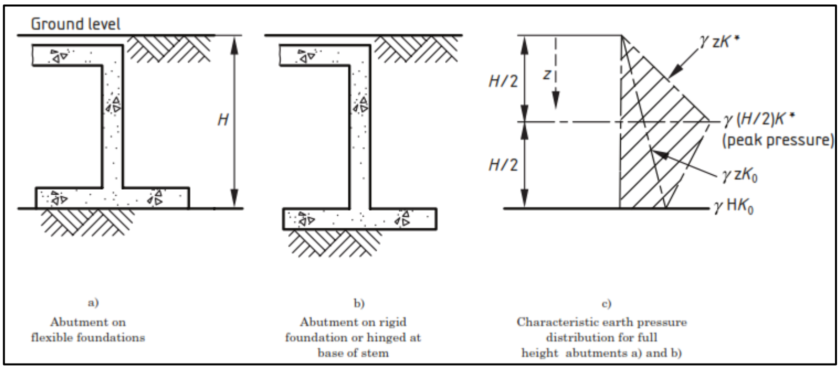

Figure 13 – Earth pressure distributions for abutments which accommodate thermal expansion by rotation and/or flexure [6]

K.p can be found from the following Table

Table 1 [6]

$\varphi'_{triax;d}$	Values of $K_{p;t}$				
	Inclination of abutment face				
	Vertical	Forwards		Backwards	
		+10°	+20°	−10°	−20°
30°	4,29	3,67	3,15	5,00	5,79
35°	5,88	4,86	4,02	7,09	8,49
40°	8,38	6,65	5,28	10,51	13,06
45°	12,57	9,51	7,20	16,52	21,45
50°	20,20	14,24	10,28	28,10	38,55

NOTE Values of $K_{p;t}$ are the horizontal component of K_p with $\delta_d/\varphi' = 0{,}5$ calculated from the equations given in BS EN 1997-1:2004, Annex C using the design values of the triaxial φ'.

2.2.3.2 Horizontal earth pressures on end screen and abutment that accommodate thermal movements by translation without rotation

For abutments that accommodate thermal movements by translation without rotation, the soil pressure may be calculated from the following equation.

$$K^* = K_0 + \left(\frac{40\, d'_d}{H}\right)^{0.4} K_p$$

This type of abutment, includes, bank pad and semi-integral end screen abutments and sleeved piles.

In the sleeved piles, the top 4 to 6 m will be isolated from the soil by an isolation device. In this type of abutment, the pile is flexible and only the end screen, which is attached to the end of the deck moves into the fill.

The isolation may be achieved with the use of a durable steel pipe, or concrete pipe (e.g. manhole rings), or plastic pipe (all with a void therein).

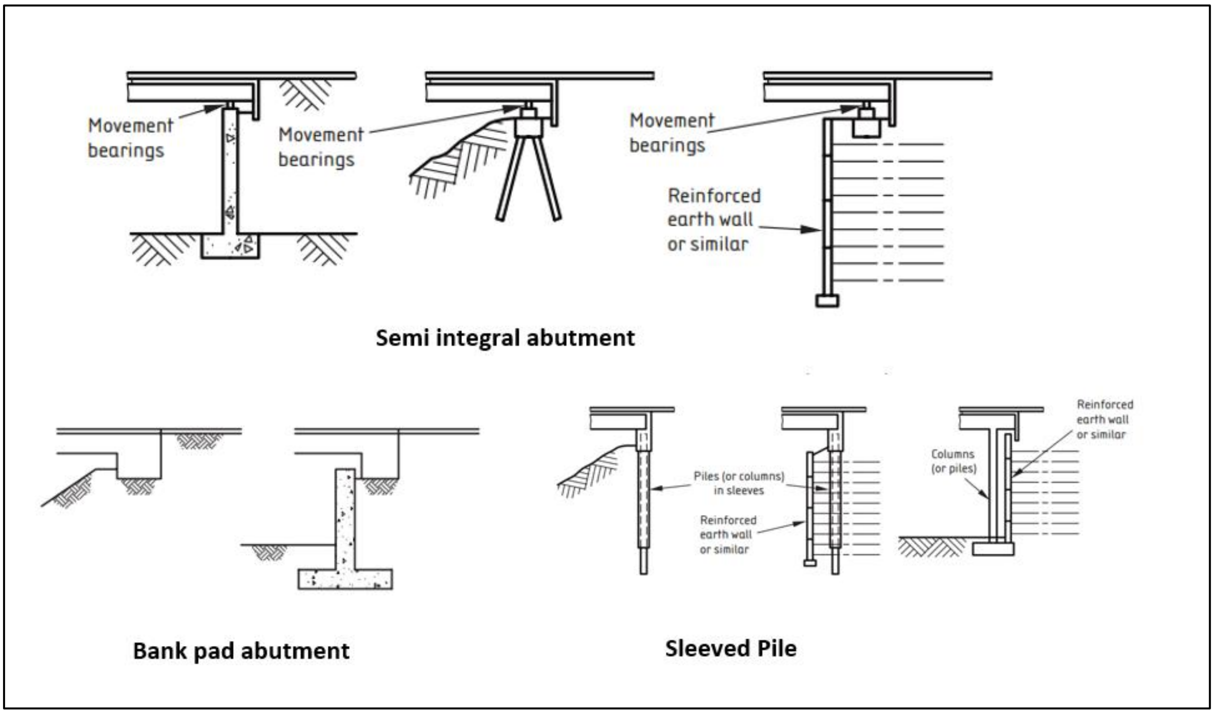

Figure 14 – Earth pressure distributions for abutments which accommodate thermal expansion by translation without rotation [6]

2.2.3.3 Horizontal pressure on abutments founded on piles

In some design guidelines [1] it is recommended that the piles are isolated from the soil by an isolation device. This would make the substructure flexible and would reduce the soil ratchetting effect. However, according to PD6694 [6] founding abutments on a single or double row of piles is permitted. However, the soil pressure due to soil ratchetting would increase and the structure should be designed for a higher soil pressure load as is discussed in this section.

For Ashton Avenue Bridge initially, the abutments were designed to be supported on a single row of 600 mm diameter concrete piles. However, due to limited space behind each abutment, using pile rigs capable of this size piles were unable to fit. Hence, in order to be able to use a smaller rig, two rows of 450 mm diameter piles were considered for the foundation, as shown in Figure 15.

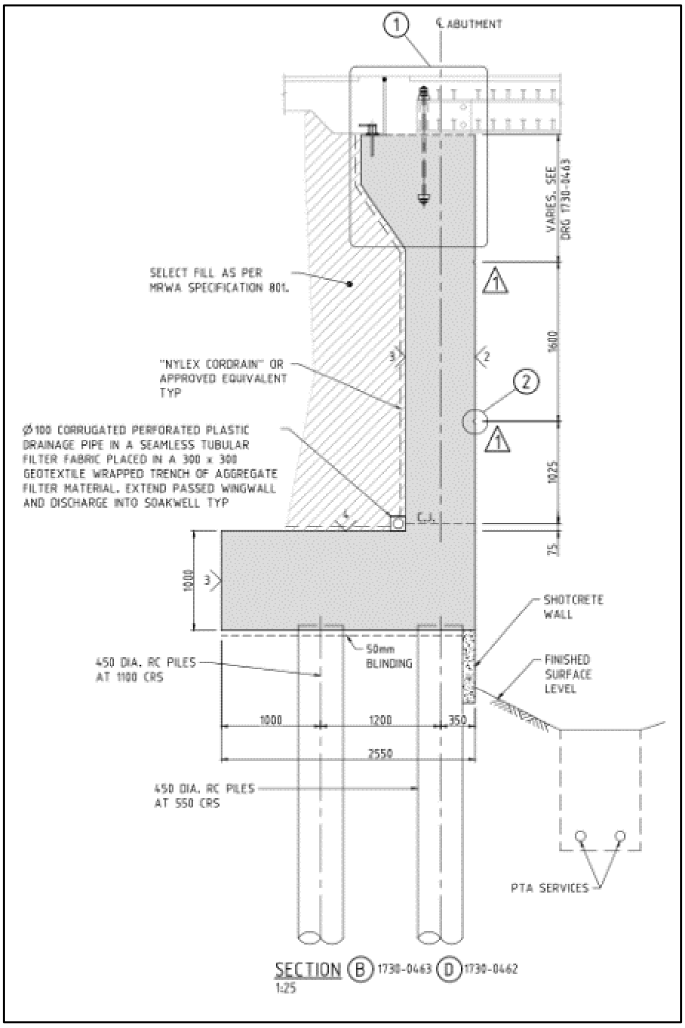

Figure 15 – Ashton Avenue Bridge Piled foundation

To calculate the soil pressure on piled footing an iterative approach is required. The distribution of soil pressure behind and in front of the abutment is shown in Figure 16 & Figure 17

The quasi-passive limiting pressure coefficient K* was calculated based on the following formula:

$$K^* = K_0 + \left(\frac{C\, d'_d}{H}\right)^{0.6} K_p$$

For the back of wall (See Figure 16), H' is the depth of soil behind the abutment affected by the repeated deck expansion, which may be taken as the depth from ground level to the level at which the earth pressure reduces to its at rest value when the deck is at its maximum expansion for the combination of actions under consideration. And *d*' is the corresponding horizontal deflection of the abutment wall at a depth *H*'/2 below ground level.

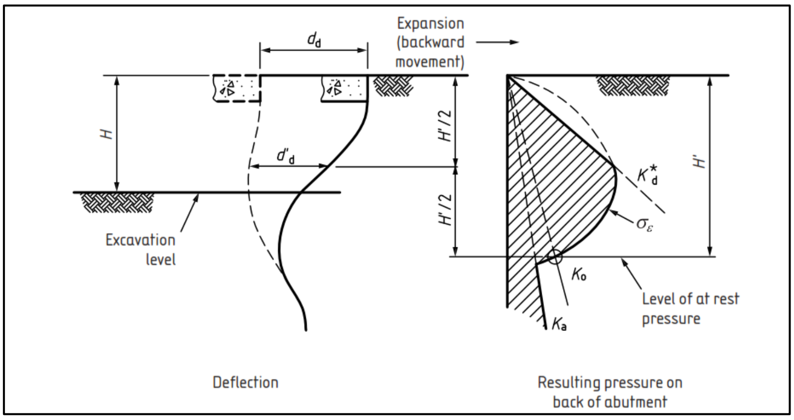

Figure 16 – Earth pressures applied to the back of abutment (expansion case) [7].

For the front of the abutment (see figure 17) *H*' is the depth of soil in front of the abutment affected by the repeated deck contraction. It is taken as the depth from excavation level to the level at which the earth pressure on the front of the wall or piles reduces to its at rest value when the deck is at its maximum contraction for the combination of actions under consideration. *d*' is the corresponding horizontal deflection of the abutment wall at a depth *H*'/2 below excavation level

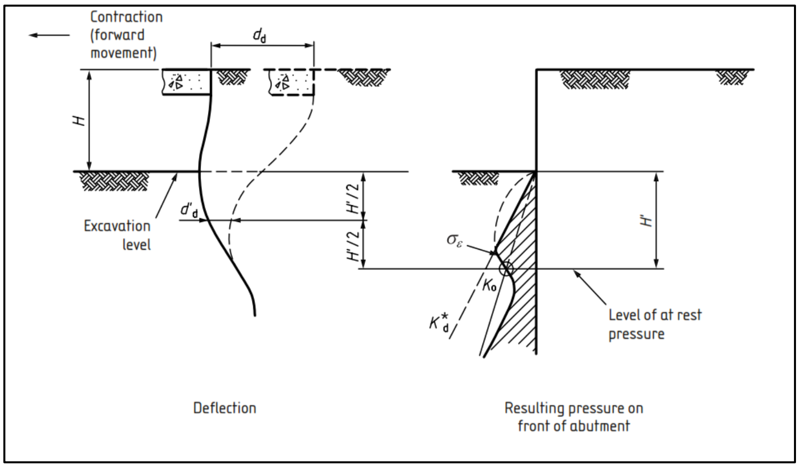

Figure 17 – Earth pressures applied to the front of abutment (contraction case) [6]

For deriving d'_d and *H*', the following iterative procedure was used.

- Step 1: The soil pressure behind the abutment was calculated based on the movement due to maximum expansion of the bridge and assuming H'=H & d'=0.5 d_d .
- Step 2: The soil pressure in front of piles was calculated based on the movement due to maximum bridge expansion and assuming: H'=H & d'=0.1 d_d .
- Step 3: The soil was modelled as series of springs. The springs stiffness's (E.d) were calculated using the following formulas:

$$\sigma_v = \gamma \cdot z - u$$

$$\sigma_m = \frac{\sigma_v \cdot (1 + 2K_0)}{3}$$

$$G = 220 \cdot \frac{R_{FG}}{\sqrt{\sigma_m}}$$

$$E_d = 3G(1 + \nu)$$

where:

- σv is the vertical stress;

- σ.m is the mean stress;
- u is the hydrostatic pressure;
- G is the shear modulus of the soil;
- ν is Poisson's ratio (which may be taken as 0,3);
- RF,G depends on the average rotational strain d'/H'(see Figure 18);

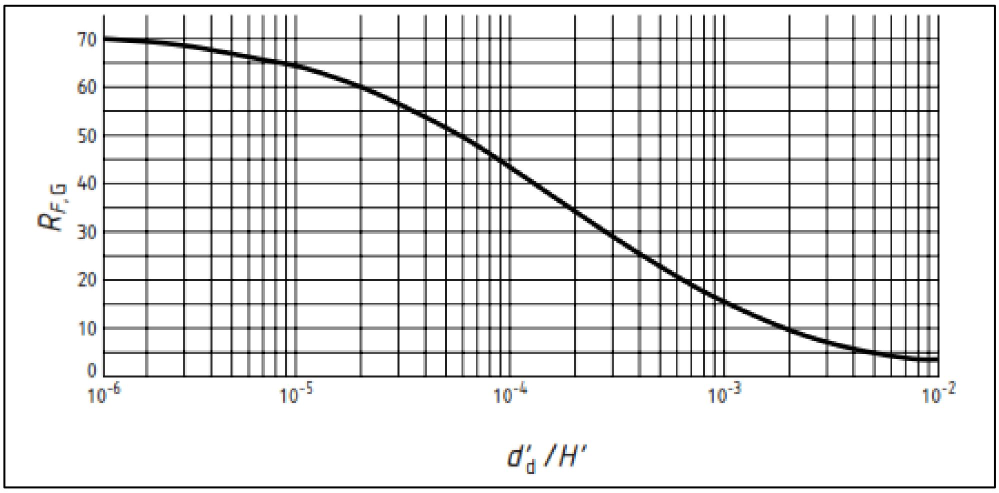

Figure 18 – Variation in soil shear modulus factor [7]

- Step 4: Apply contraction giving a movement of $d._d/2$ at the end of the deck;
- Step 5: Apply expansion giving a movement of $d._d$ at the end of the deck and determine earth pressures applied to the abutment for this expansion case;
- Step 6: Compare the values of *H*' and *d*' for the back of the abutment with those used in Step 3. If these are significantly different, repeat Steps 4 to 6 using updated values of *H*' and d'_d;
- Step 7: Apply contraction giving a movement of *d* to the end of the deck and determine earth pressures applied to the abutment for this contraction case;
- Step 8: Identify the depth below excavation level at which the earth pressure in front of the abutment reduces to at rest pressure (*H*') and the resulting deflection *d*' at *H*'/2; and
- Step 9: Compare the values of *H*' and d'_d for the front of the abutment with those used in Step 3. If these are significantly different, repeat Steps 7 to 9 using updated values of *H*' and *d*.
- *d*'

2.2.4 Creep and shrinkage [9]

The remaining force is the differential shrinkage caused by the time delay between casting the main girders and placing the deck. The creep and shrinkage shall be considered in designing the superstructure; however, it can be ignored in designing the substructure. As the main girders and slab shorten due to shrinkage, the applied soil pressure will be active soil pressure which is far less than the soil pressure due to soil ratcheting (K*). Therefore, the overall shortening due to shrinkage and creep does not need to be considered in the design of integral bridge substructure. However, the bending moment due to shrinkage and gradient temperature should be considered for designing the pile.

2.2.5 Backfill material

Backfill material type has a significant impact on induced soil pressure. Generally, the built-up soil pressure due to ratcheting effect is a function of backfill material's modulus of elasticity.

The result of a survey conducted by Utah university [12] shows that the most common backfill material both in Europe and in the United States is well-compacted gravel or sand. In the United States, 69% of State agencies required a well compacted granular backfill, while 15% required the backfill to be loose in an effort to reduce forces on the moving abutment stem. None of the European countries required the use of an elastic material behind the abutments. However, in the United States, 16% of respondents indicated the use of some type of compressible material (Geofoam, or polystyrene) behind the abutment stem.

In this project, a compacted granular material with a maximum of 10% fill material passing 75μm sieve was selected. The expected modulus of elasticity was 70MPa.

During the construction the backfill was placed simultaneously behind both abutments at the same time, the difference in elevation was limited to 0.5 m.

2.3 Construction sequence

The following outlines the construction sequence adopted:

1. Deconstruct the existing bridge, and install temporary sheet piling.
2. Install piles and pile cap.
3. Construct the first stage of the abutment structure up to 25 mm lower than slab soffit.
4. Install the Macalloy bolts and sacrificial bearing pads.
5. Land the planks on sacrificial bearing pads and grout the gaps between the planks. This was done during the weekend rail shutdown period;
6. Cast the first stage of the deck pour - a strip 10 m long at the midspan of the deck;
7. Tighten the Macalloy bolts (snug tight) and cast the second stage of deck;
8. Cast the final stage of the deck(third stage);
9. Grout the space under the girders at the abutments using a flowable grout;
10. Construct the abutment wingwalls;
11. Install abutment drainage system;
12. Install special integral abutment backfill in balanced lifts; and
13. Complete the approach slabs.

3 CONCLUSION.

Battledeck planks feature high robustness and low maintenance in service, which meet the demand for our infrastructure of the future. Known as a traditional solution, however new products as new heavy rolled section with higher steel grades and enhanced construction methods offer new possibilities for this deck type. Combining the battledeck superstructure with integral abutments will reduce the section size considerably and it can be a smart solution for the crossings where depth is critical.

4 REFERENCES:

1. NJDOT Design manual for bridges and structures – 5th Edition Integral Abutment Jointless Bridges (US Design Guide Line (Chapter 15))
2. Design manual for roads and bridges (DMRB). London: The Stationery Office. “Volume 1: Section 3”
3. General Design, BD42/96- Amendment No. 1- The Design of Integral Bridges”.
4. Bridge Technical Note 2012003V10
5. Earthquake resistance of integral abutment bridges FHWA/IN/JTRP-2008/11

6. Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004 pd 6694
7. Draft technical report on integral bridges SCI 340
8. AS 5100.6:2017
9. EN 1994.1.2 Design of composite steel and concrete structures Part. 1-2
10. EN 1994.1.1 Design of composite steel and concrete structures" Part 1"
11. EN 1994.2 Design of composite steel and concrete structures Part. 2
12. Behaviour and Analysis of an Integral Abutment Bridge- Utah university-2013.
13. Structural behaviour of Battledeck Floor system. By Inge Lyse M .American Society of Civil Engineering 1938.

5 ACKNOWLEDGEMENTS

I wish to acknowledge my colleagues at GHD.Pty.Ltd who conducted the design review and assisted to the preparation of this paper. I especially thank Liz Tinlin for her support during the design and for sharing her insight expertise that greatly assisted through this project and her comments that significantly improved the manuscript.

I also would like to show my gratitude to Peter Tonkin for his support to attend this conference.

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Behzad has been involved in the delivery of the projects, including project management, tendering, contract administration and provision of technical support during construction works.