Integral Abutment Bridges – Australian and US Practice

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SYNOPSIS

The structural system offered by bridges made integral between superstructure and abutments can provide structural efficiencies as well as enable the elimination of bearings and expansion joints. In some circumstances the durability of the bridge is improved and maintenance costs reduced. The benefits to be gained are greater in more severe climates and under more severe loading conditions.

The use of integral abutment bridges is not widespread in Australia where climatic conditions are relatively benign. However there are a number of examples of integral abutment bridges, and their design and the typical Australian practice is illustrated along with a particular case study.

Integral abutment bridges are more widespread in the USA, and have been used for many years. The frequent use of integral abutments and their reported good performance has led to a number of US Departments of Transportation developing standard details and design guidelines.

The performance of integral abutment bridges in the US is described by reference to the literature and in particular to surveys of US Departments of Transportation. The typical limits on bridge length, skew and thermal ranges are indicated, based on the average practice of the departments.

Particular design issues are discussed based on past performance of integral bridges in Australia and US practice.

1 INTRODUCTION

Integral abutment bridges can be described as bridges generally built with their superstructures integral with the abutments, and without expansion or contraction joints for the entire length of the superstructure. The abutments, being cast integral with the superstructure, avoid expansion joints and movement bearings that otherwise require regular maintenance.

The benefits of integral bridges are principally the elimination of expansion joints and some bearings, leading to simpler structures that are easier and less expensive to maintain. However the complex interactions between integral abutment elements and the soils that surround them are difficult to establish with precision. In fact there are no straightforward techniques for quantifying the soil-structure interaction. This is perhaps the reason for some past reticence in adopting this form of construction, and also for a more empirical approach to their design.

This paper indicates the benefits in adopting integral bridges and explores the limited practice and use of integral bridges in Australia. Integral bridges are more widely used in North America and the US practice is presented, drawing upon the developed ideas of the State Departments of Transportation. Some indication of the performance of integral bridges is provided by reference to an Australian example and American literature.

2 BACKGROUND

It is the "whole of life" cost and maintenance considerations that have led bridge engineers to the use of integral bridges. Expansion joints have long been one of the most exposed and severely loaded elements in bridge structures. They are loaded heavily by axle and individual wheel loads, and subject to the accumulation of dirt and grit, and to the extremes of thermal movements. In areas where low extremes of temperature are experienced, the use of de-icing salts adds a further environmental 'load', with the resulting acceleration of deterioration of the expansion joint, bearings and possibly areas of the main structural members. The cost of maintaining and repairing all these elements is due not just to the physical work involved, but also the disruption to traffic with road closures. These costs are of course magnified for toll road bridges.

The economic drivers for using integral bridges are emphasised when the bridges need to endure low extremes of temperature, because of the prevalent use of de-icing salts in such situations. Accordingly, the practice of incorporating integral abutments in bridges is more widespread in the US than in Australia.

Except for minor roads in snow resort areas, Australian roads rarely experience snow or ice conditions and the use of de-icing salts is not necessary. There are therefore no documented guidelines for the design of integral bridges in Australia. They must be designed using good engineering practice and generally by reference to American or British practice.

Whilst the decision to adopt integral bridge abutments and jointless bridges may be driven by the benefits in maintenance and bridge repair, other benefits can ensue:

- In some instances the deletion of bridge bearings and expansion joints and their replacement with built-in connections can save initial costs.
- Built-in abutments can be designed to accommodate some bending moment capacity, reducing end span bending moments with possible minor savings in end span girders.
- If end spans are part of a flexurally continuous superstructure, integral abutments can provide capacity against uplift if necessary.
- Bridge bearings usually require a reasonably close degree of tolerance in their assembly. This precision can be avoided with integral abutments.

- A jointless bridge with integral abutments will have a greater degree of redundancy that may be beneficial in earthquake zones. The problems of retaining the superstructure on its bearing shelf during seismic events is eliminated and the inherent damping of the integral bridge structural system allows it to better absorb energy and limit damage.

3 AUSTRALIAN PRACTICE FOR INTEGRAL ABUTMENT BRIDGES

The Australian Bridge Design Code (1) contains no particular reference to integral abutment bridges or jointless bridge decks. Designers must refer to the general design requirements contained in the code and to relevant specialised literature.

Basically the bridge structure and integral abutments in particular must be designed for the movements due to thermal strains, both increasing and decreasing, and the decreasing strains due to shrinkage of the superstructure if it is concrete, plus the decreasing strains due to creep of prestressed concrete decks. The forces developed in the structural system are a function of the relative stiffness of the elements, their foundations and their interaction with the surrounding soils.

The forces due to applied loads must also be accommodated, and these are the vertical dead loads, the vertical loads due to vehicular live loads plus impact, the horizontal loads due to wind, vehicle braking and centrifugal loads and seismic loads if applicable.

The connection between the superstructure and the substructure must also be designed for a Minimum Lateral Restraint Capacity to ensure that the superstructure has sufficient lateral restraint to resist unaccounted lateral forces not otherwise catered for in the design. Integral abutments are therefore also designed for a force of 500kN or 5% of the superstructure dead load, whichever is the greater. This force is considered an Ultimate Limit State force.

A new Australian Bridge Design Code (2) is currently in draft form and is expected to be published during 2004. There are currently no plans to include any reference to integral bridges in this new code. This code does however increase live loading significantly, and both the vertical effects and the longitudinal braking effects of vehicle loading are increased. Both these loads will have an impact on the design of future integral abutment bridges.

The key design movement conditions indicated in the current Austroads (1) code that pertain to the design of integral bridges are listed below.

3.1 Thermal Movements

Bridges must be designed for the movements that result from the temperature effects indicated in Table 1. These ranges depend on the bridge location and the widest range is indicated. The movements determined from these temperature ranges are Serviceability Limit State values. They are multiplied by a factor of 1.25 to convert them to Ultimate Limit State values. Using a co-efficient of thermal expansion of 11x

10^{-6} per degree Celsius, these thermal movements can range between 500 and 850 microstrain depending on bridge type and location.

	Average Bridge Temperature ^{0}Celsius		
Bridge Type	Minimum	Maximum	Range
Concrete Superstructure	3	50	47
Concrete Deck on Steel Beams	-2	60	62
Steel Superstructure	-7	70	77

Table 1: Thermal Ranges for Bridge Design – Austroads (1)

3.2 Concrete Shrinkage

Shrinkage varies due to the concrete properties and its basic shrinkage parameters, the thickness of the concrete elements and their area exposed to drying conditions, as well as the prevailing climatic conditions. Average shrinkage strains can result in 250 to 400 microstrain contraction.

3.3 Concrete Creep

Creep in prestress concrete superstructures can range considerably depending on the concrete properties and level of prestress. Creep strains can range up to 400 microstrain.

The effects of creep and shrinkage are multiplied by a load factor of 1.2 to convert them to an Ultimate Limit State effect. The combination of thermal effects plus creep and shrinkage typically results in ultimate limit state strains of 1000 microstrain contraction and 450 microstrain expansion, for prestressed concrete superstructures.

If the structural system of the deck, piers, abutment wall and abutment piles is comprised of concrete, the modifying effects of creep are considered when determining forces and stresses in the concrete determined using an elastic analysis. An effective elastic modulus approach can be adopted to determine the modifying effect of creep on the peak elastic stresses.

4 INTEGRAL ABUTMENT BRIDGE DETAILS

The typical Australian practice for most medium to short span bridges is to minimise the use of expansion joints generally by adopting link slabs over piers for simply supported spans (Figure 1). This practice is described in detail in Kumar (3). The use of link slabs provides a means of transferring force but the low bending stiffness of the link slab ensures effectively no moment continuity at the pier. The resulting bridge deck is “jointless” between expansion joints that break the bridge up into longer segments of superstructure.

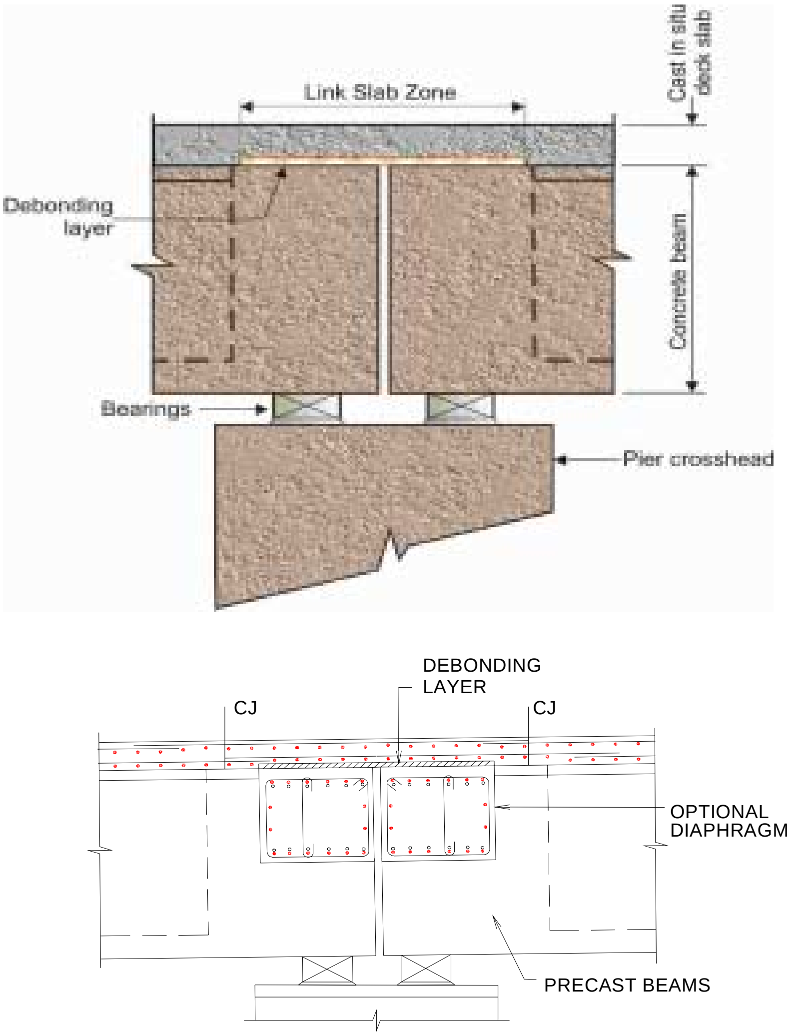

Figure 1: Link Slab Detail at Intermediate Pier Locations

The same practice is adopted for integral abutment bridges with the deck slab being jointless between abutments. If full fixity is established between the superstructure and the abutments, these bridges are deemed "fully" integral. Fixity is established by ensuring the deck and beams are cast integrally with the abutment wall. The beams will have reinforcement protruding from their ends, and the deck reinforcement is turned down into the abutment diaphragm pour to achieve the moment connection. The typical abutment detail for these bridges is shown in Figure 2.

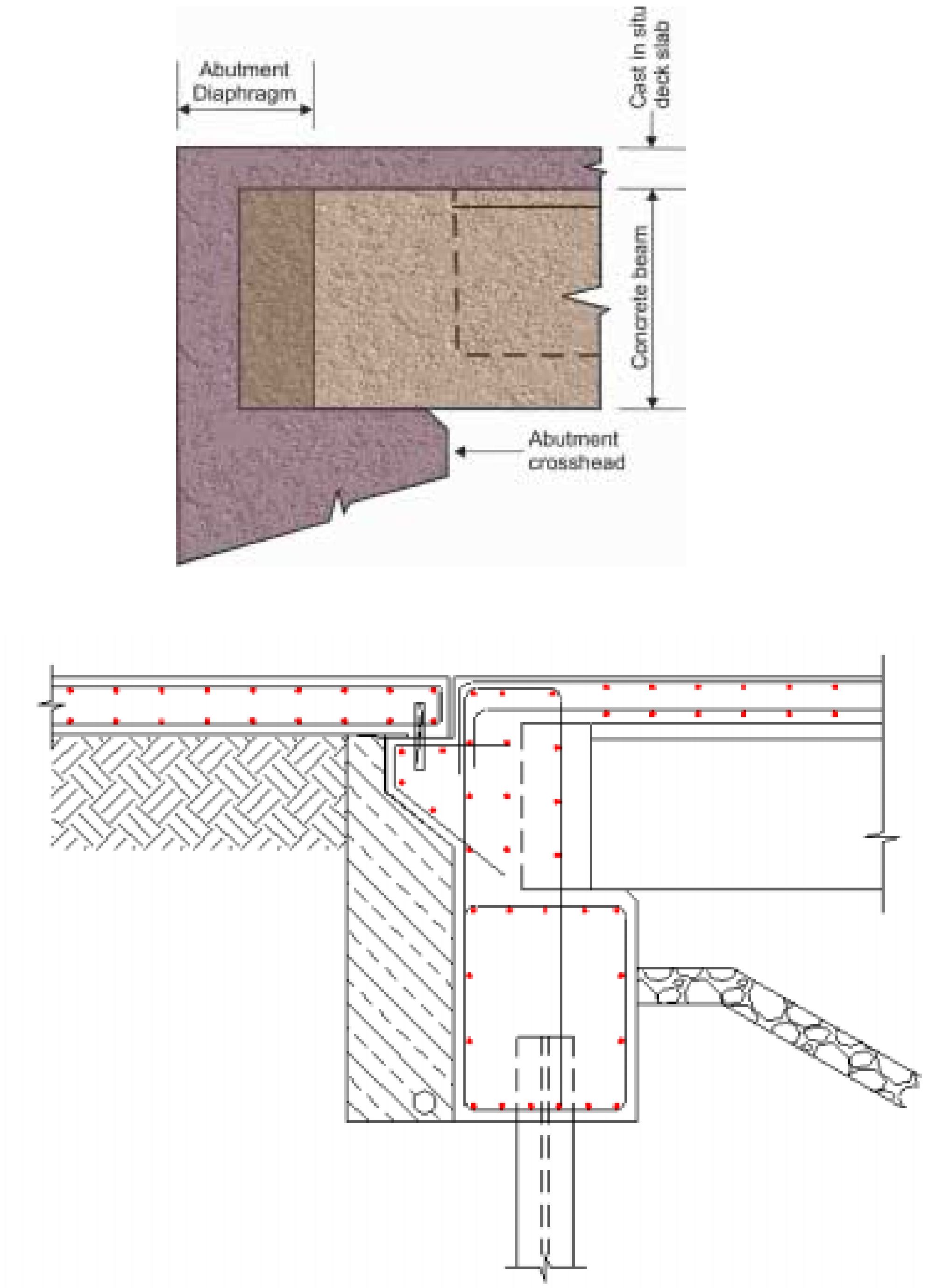

Figure 2: Fully Integral Abutment Details

The "fully" integral connection between the beam and abutment is adopted if the effect of continuity is deemed beneficial. Adopting a moment connection will reduce the bending moment in the span with possible savings in beam depth and capacity. However the savings in the beam can be offset by ensuring adequate moment capacity at the beam/abutment junction. Often the lap lengths and reinforcement congestion that ensues makes these connections of marginal value.

If full continuity at the abutments is not worthwhile, a "semi-integral" abutment can be adopted. In this case the link slab arrangement is adopted throughout and also at the abutments. In these cases elastomeric bearings are usually installed between the main beams and the substructure to transfer vertical force but allow rotation of the deck relative to the pier or abutment. (Figure 3)

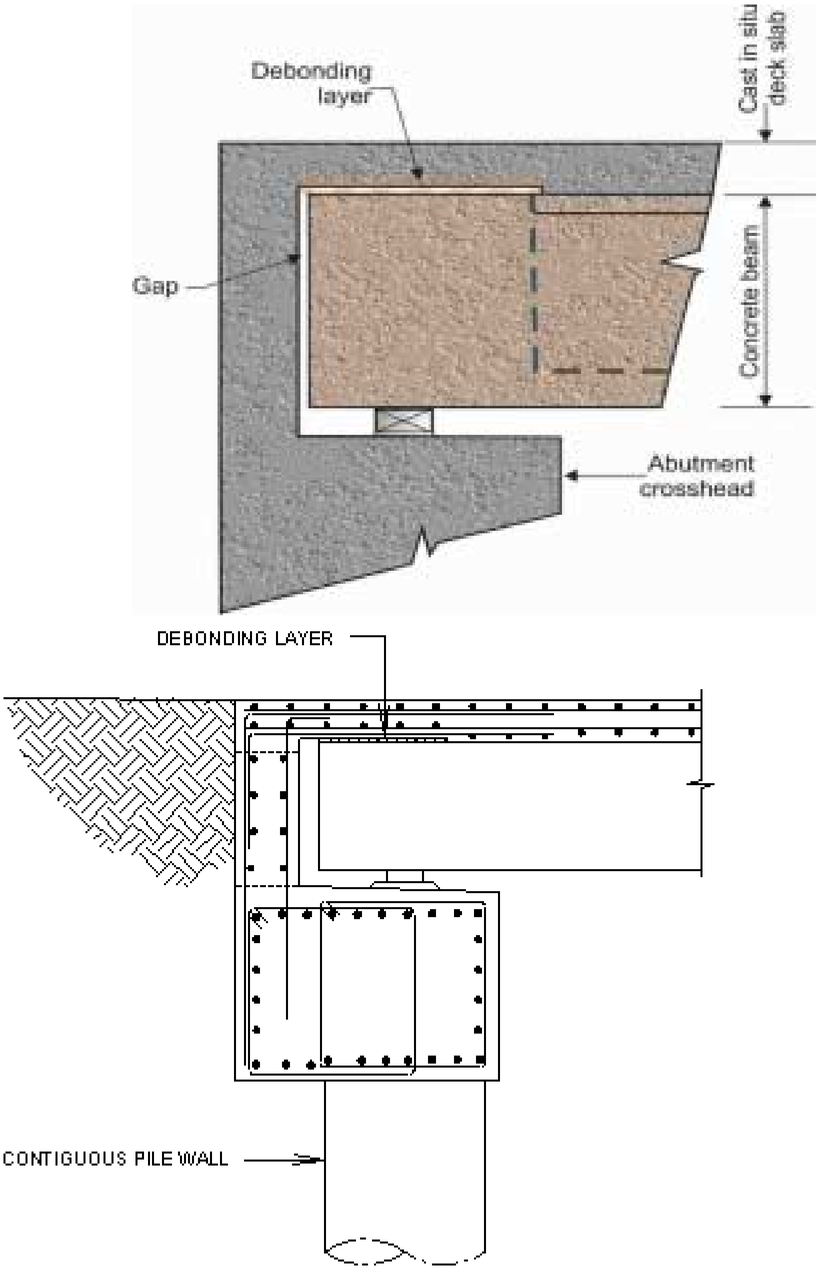

Figure 3: "Semi-integral" Abutment Details

Integral abutments are usually supported on vertical piles, typically with spill-through abutments. Piles are designed to support the necessary vertical loads yet retain sufficient flexibility such that forces due to movements are minimised. This usually results in a single line of piles with no raked piles. Ideal foundation conditions are where piles may be founded on rock at depth to achieve vertical capacity, but pile sections are slender to limit flexural stresses. Precast concrete piles have been adopted, as have steel 'H' piles with their minor axis of strength perpendicular to the bridge centreline such that they flex about their minor axis.

Where integral abutments have been adopted in underpass structures with vertical walls, a single line of contiguous bored piles has been frequently used. These structures are usually shorter spans, where the stiffer contiguous pile wall is not subject to deck movements that induce intolerably large flexural stresses in the bored piles or large tensions in the deck. The connection arrangements for these structures can be either pinned or fixed, depending on the ease with which a moment connection can be achieved between the deck and the contiguous pile wall.

5 AUSTRALIAN EXAMPLE – GILLIES STREET BRIDGE

A two span continuous bridge was constructed as part of the Ballarat Bypass project in 1995. The bridge is continuous over a central pier with two spans of 29.5m and a 10 degree skew. The superstructure comprised tee girders with a cast-in-situ concrete deck slab. The girders were built into the central pier and both abutments giving a jointless deck and integral abutments. The bridge is pictured in elevation in Figure 4, and shown in general arrangement in Figures 5 and 6.

Figure 4: Elevation of Gillies Street Bridge

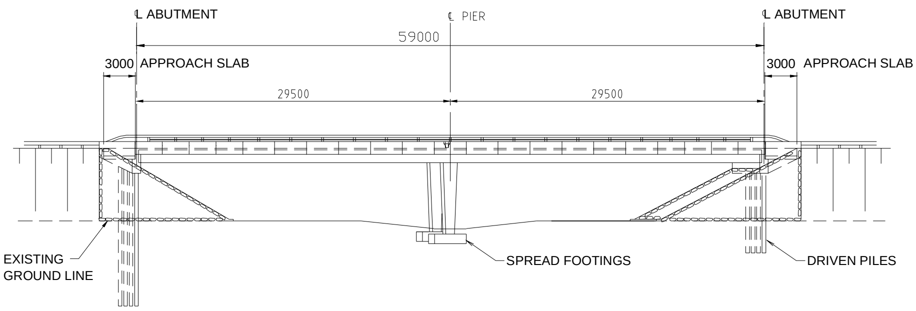

Figure 5: Gillies Street Bridge – General Arrangement

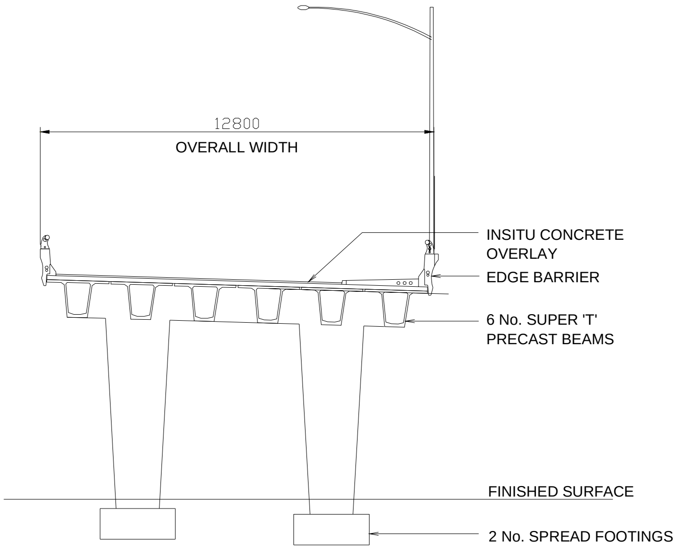

Figure 6: Gillies Street Bridge – Cross Section

5.1 Design Parameters

The bridge was designed as an integral abutment bridge with moment fixity between the superstructure and the abutments. The design parameters for the bridge are summarised as follows:

- Design to Austroads (1)
- Live loading T44 truck loading and L44 lane loading
- Serviceability Limit State movement strains

Temperature rise	350 microstrain
Temperature fall	190 microstrain
Shrinkage	350 microstrain
Creep	290 microstrain

- Total Ultimate Limit State contraction 1000 microstrain
- Total Ultimate Limit State expansion 440 microstrain
- Vehicle braking force 1200kN, distributed 500kN to the central pier and 700kN to the abutments.

5.2 Design Approach

The thermal strains of the superstructure were determined based on an initial ‘closing’ temperature of 20 degrees Celsius. The creep and shrinkage strains recognised the age of the precast beams at the time of closing the joint between the superstructure and the abutment. The long-term movements due to creep and shrinkage of the deck cause permanent deformation of the structural system and flexure in the reinforced concrete abutment piles. The resulting stresses were modified and reduced to account for the long-term creep effects in the piles

The backfill at the abutment was a structural granular material with an angle of internal friction, $\Phi=30\text{-}35^0$ with negligible cohesion. This is maintained in a drained condition using a drainage layer at the rear of the abutment. The piled abutment was analysed using a frame model and assigning springs to represent the effect of the backfill material. The stiffness of the springs recognised the drained nature of the backfill.

The particularly high vehicle braking loads that must be resisted by the abutments were resisted by a combination of a buried approach slab that mobilises friction within the approach embankment, the flexure of the abutment piles and by the passive resistance generated at the rear of the abutment crosshead. The detail of the buried approach slab and integral abutments is shown in Figure 7.

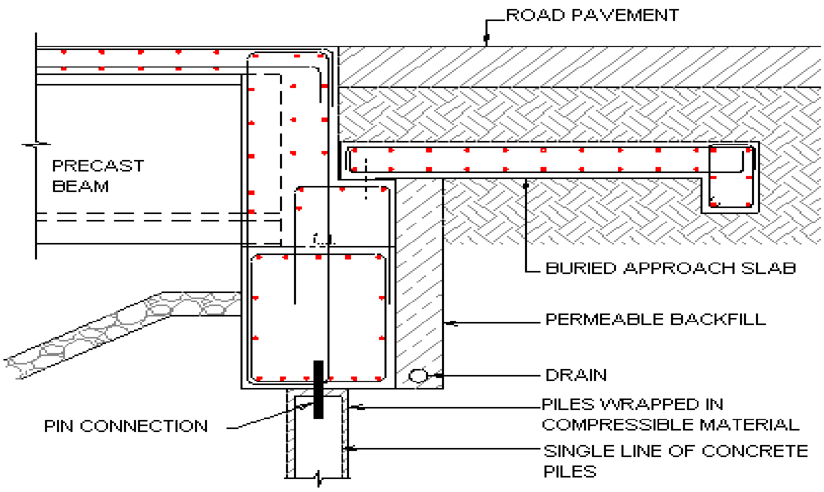

Figure 7: Buried Approach Slab and Integral Abutment

The connection between the abutment piles and the abutment crosshead is pinned. This allows only vertical and shear forces to be applied to the tops of the piles. No bending moments are transferred from the superstructure to the pile tops. The piles were wrapped with compressible material to ensure minimal lateral forces are developed in the upper sections of the piles under thermal and other strains in the deck. The pile details are indicated in Figure 8.

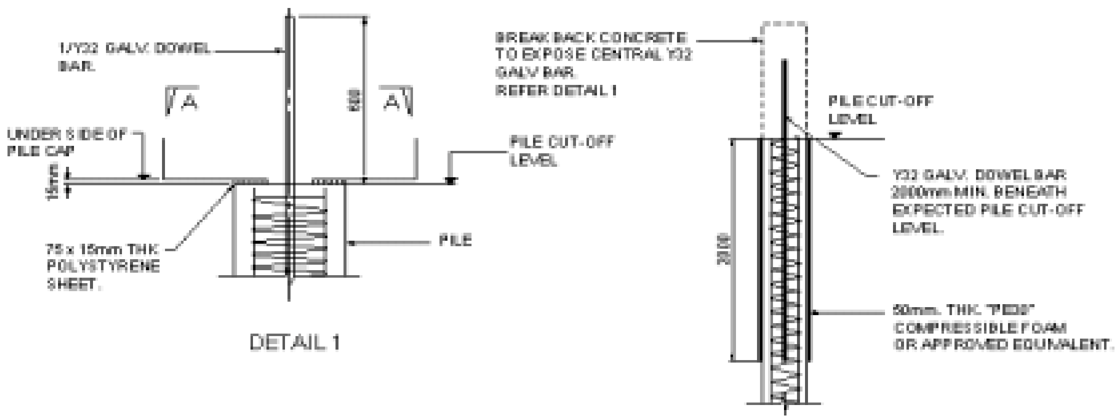

Figure 8. Pile Details

5.3 Construction Sequence

The approach embankments were first established up to the level of the underside of the abutment crosshead. The abutment piles were driven to their founding level onto rock with pre-boring of the upper 2 metres.

The pinned connection between the piles and the abutment crosshead was then prepared, and the abutment crosshead cast up to the level of the underside of the beams. Meanwhile the central pier with its spread footing foundation was also constructed.

The bridge superstructure was then constructed by supporting the bridge beams on falsework, and casting the central pier diaphragm and deck slab to form the continuous superstructure.

The final construction stages involved casting the rear wall of the abutments around the ends of the beams, and making a moment connection between the deck and the abutment. The approach slab and wing walls were then cast, and the remaining approach earthworks and pavement completed along with rock beaching on the face of each abutment slope.

5.4 Bridge Performance

The bridge has been in service for approximately 7 years and is subject to a high percentage of truck traffic. The site of the bridge is relatively exposed and the climate is regarded as inland temperate. Extremes of air temperature at this location range from -5^{0}Celsius to 45^{0}Celsius.

The bridge deck is in excellent condition and shows no signs of deterioration (Figures 9 and 10). The central pier is also in similar condition.

Figure 9: Bridge Deck View

Figure 10: Underneath View Showing Central Pier

The integral abutments show no signs of cracking or movement that would indicate distress or any inability to accommodate the movements or flexure imposed from the superstructure. Figure 11 shows no cracking of the abutment elements, and there is little or no movement evident at the front of the abutments. The grouted rock beaching is undisturbed and has few cracks (Figure 12). There is some evidence of deck contraction at the end of the abutments (Figure 13) and this measured less than 10mm on a cold day in mid winter.

Figure 11: Abutment Elevation

Figure 12: Front of Abutment Showing Condition of Rock Beaching

Figure 13: View Showing Gap of Less Than 10mm at End of Abutment

The approach road surface and the deck surface have a good ride quality and there has been no discernable settlement of the approach embankments. There are however some repairs to cracks that have appeared at the interface between the abutment rear walls and the road pavement, above the buried approach slab. This has been repaired by the Road Authority (VicRoads), by installing a rubberised seal. A further crack in the approach pavement has also opened slightly in the vicinity of the end of the approach slab and this has been filled with bitumen as a joint gap filler to seal the crack against moisture penetration (Figure 14).

Figure 14: Approach Pavement Crack and Joint Filling

The overall performance of the bridge has been very good, however there has been some attention to pavement cracks behind the abutments. These cracks have not led to further deterioration or other structural problems, as they have been sealed to prevent moisture ingress to the pavement layers, and in any case the zone behind the abutments is well drained.

The conclusions to be drawn from the condition of this bridge are:

- Prestressed concrete bridges in the length range up to 60m can perform well as integral bridges.
- The effects of creep and shrinkage induce a prevailing shortening of the bridge superstructure of prestressed concrete integral bridges. This shortening is greater than the expansion effects due to temperature increases.
- The cyclic effect of thermal movements is not structurally significant.
- The abutment movements need to be accommodated. This appears better addressed using an approach slab at pavement level with a control joint at the end of the approach slab, rather than by a buried approach slab.

6 AMERICAN INTEGRAL BRIDGE PRACTICE

The American State Departments of Transportation have adopted the practice of jointless bridge decks and integral abutments as early as the 1930's. Burke (4), in a National Co-operative Highway Research Program report summarised the results of a 1973 study of integral construction and conducted a survey of transportation departments about their use of integral construction for continuous bridges. The survey found that:

> *"Bridges with integral abutments suffered only minor damage from pavement pressure, were essentially unaffected by de-icing chemicals, and functioned for extended periods without appreciable maintenance or repair. Moreover, they were usually less expensive to construct"*

The Burke (4) survey had 20 responding transportation departments and 67% of those indicated they are now using integral construction. This use started in the 1930's but gained momentum in the 1960's and 1970's. There is a range of design details that have been adopted in the US over this time and a good pool of experience to draw upon.

In more recent times, Kunin et al (5) reported on the current United States and Canadian practice on integral bridges. This report focused on New York State practice but surveyed 39 State or provincial transportation agencies in the United States and Canada. The key findings of this more recent survey were:

- The longest precast concrete girder integral bridge was 358.4m long.
- The longest steel girder integral bridge was 318.4m long.
- The longest cast-in-place concrete bridge with integral abutments was 290.4m long.
- Design temperature ranges varied slightly between departments but in general were:
    - -18^{0}Celsius to 50^{0}Celsius for steel bridges in moderate climates
    - -35^{0}Celsius to 50^{0}Celsius for steel bridges in cold climates
    - -5^{0}Celsius to 32^{0}Celsius for concrete bridges in moderate climates
    - -10^{0}Celsius to 36^{0}Celsius for concrete bridges in cold climates
- About 75% of respondents indicated no design allowance for shrinkage
- Those agencies that accounted for creep and shrinkage usually did so only for prestressed concrete bridges. Typical shrinkage strain allowance was 200 microstrain.

Many transportation agencies applied limits on the key parameters of integral bridges. The AASHTO Standard Specifications for Highway Bridges (6) has provided a lead in this regard in Article 7.5.3 when it refers to maximum span lengths indicated in FHWA Technical Advisory T 5140.13 (7), however it also allows local experience to overrule those limits. In any case the FHWA advisory has now been withdrawn and the Kunin et al (5) survey indicates a range of limits for various parameters adopted by the transportation departments as indicated in Table 2.

	Maximum allowable thermal movement (mm)	Max Bridge length (m)		Maximum Skew (degrees)
		Steel Girder Bridge	Precast Concrete Girder Bridge	
Average	49	83	118	28
Maximum	No limit	No limit	No limit	No limit
Minimum	13	24.4	18.3	0

Table 2: Ranges of the Limits on Key Parameters from US Practice

The widespread use of integral bridges in North America is an indicator of their generally good performance. Indeed numerous state transportation departments have developed standard details and design guidelines for integral bridges. These have been developed over many years experience in some cases. The guides they provide are often based on past practice, rather than on detailed analytical work. However this is not surprising, given the complex soil-structure interaction that occurs between abutment crossheads, piles and the surrounding approach earthworks.

In the Kunin et al (5) survey numerous respondents made comments on the performance of integral bridges. Some of the relevant comments are:

- About half the designers reported no distress related to thermal movement. Where there were problems, these were usually limited to cracking and approach slab settlement.
- Most respondents reported no unexpected behaviour with respect to thermal movement.
- Most agencies rated the performance of approach slabs at least satisfactory. The typical defects related to settlement, some slab cracking and cracks in asphalt overlays at the ends of approach slabs.
- Bridge owners rated the performance of their integral abutment bridges as “good” or “excellent” and have experienced only minor problems in general.

7 DESIGN ISSUES

7.1 Continuity and Abutment Fixity

Jointless bridge decks can be achieved either by continuous bridge superstructures that develop continuity bending moments over the internal piers, or by the use of link slabs over piers that simply link the deck slabs of simply supported spans. The outcome in terms of the bridge response to thermal movements and applied longitudinal loading is similar, and the design of the integral abutments at the ends of the bridge essentially the same, regardless of the arrangements at intermediate piers. The choice of full continuity or simple spans with link slabs is made on the basis of the economy of the superstructure design.

The issue of continuity at the abutment connection is more complex. Using a semi-integral abutment by adopting a link slab at the abutment connection creates a simple support of the superstructure at the abutment. In this case the flexure of the superstructure is not transferred to the abutment, and the abutment and particularly its supporting piles are not subject to load repetitions due to vehicle live loads. The horizontal forces developed due to thermal loads or transferred due to braking loads must also be transferred in flexure through the abutment rear wall.

A fully integral abutment will reduce flexural bending moments in the end spans of the bridge, but also subject the abutment and pile system to flexure due to vehicle live loads that have a fatigue effect. The detailing to achieve flexural continuity at the abutment/superstructure junction can also result in reinforcement congestion.

7.2 Approach Slabs

The use of approach slabs connected to the abutment is widespread on integral abutment bridges. The approach slab is best located at the pavement level, rather than buried at a lower level below pavement layers. The approach slab should be a minimum of 3m long but preferably 5m long, and terminated on the same skew as the abutment.

The approach slab achieves several aims. It prevents the continuing compaction of the abutment backfill due to vehicular traffic and also provides a smooth transition for translation and rotation of the abutment and also for possible settlement of the approach embankment. The continuous connection of the approach slab to the deck and abutment rear wall also prevents water from penetrating the soils behind the abutments that can cause erosion and piping of the abutment backfill. If the environmental conditions necessitate the use of de-icing salts, the continuous connection between approach slab and deck prevents the penetration of salt laden moisture to the lower sections of the abutments and to the bearings of semi-integral abutments.

By tying the approach slab to the abutment or bridge deck, the movement of the bridge must be accommodated at the far end of the approach slab. For short bridges there is no need for an expansion joint at this location because the movements are very small. However, where movement needs to be accommodated, a control joint can be used between the end of the approach slab and the adjoining pavement, usually positioned over a small footing slab.

7.3 Abutment Piles

Abutment piles need to have sufficient vertical capacity, but low stiffness to minimise the flexural effects of thermal and other movements. For fully integral abutments it is assumed that the superstructure transfers all moments, and vertical and horizontal loads due to the full range of superimposed dead loads (those applied after moment continuity is established), live load plus impact, braking and centrifugal forces, temperature, shrinkage, creep and seismic loads if relevant.

Often a single row of piles is used with the pile oriented to ensure bending about the weak axis of the pile. If the movements are large, it is sometimes necessary to reduce the effects of the movements by pre-boring for the piles and filling the pre-bored holes with compressible material around the pile, in order to reduce the effects of high shear forces and high moments in the upper sections of the piles.

Pile moments can be further reduced if the pile is pin connected to the underside of the abutment. The pin connection then only transfers shear forces and vertical forces. Peak moments can then be reduced, and the flexure of the piles due to vehicle live loads is eliminated, with a beneficial reduction in fatigue loading.

7.4 Construction Sequence

When the abutment is made integral with the superstructure, there is a possibility that during construction the superstructure will be subject to thermal movements when the freshly placed concrete of the abutment diaphragm is not fully set. This can be a greater concern with steel beam bridges that are more responsive to rapid changes in ambient temperatures.

Casting of the closing pour of the abutment should be timed such that the initial set of the abutment diaphragm concrete occurs when the superstructure is at a relatively constant temperature. As bridge temperatures lag ambient air temperatures, casting after midnight or in the very early hours of the morning is preferred. In the case of steel girder bridges, their thermal movements are modified after casting the concrete deck slab. Therefore the best results can be achieved by casting the closing abutment joint after the deck slab has been cast, and preferably whilst the deck slab is cooled with water spraying or flooding, which is also beneficial for curing.

7.5 Skew

Integral abutment bridges are usually limited to skews less than 30 degrees. For larger skews the passive earth pressure forces generated on the abutment rear faces during bridge expansion cause lateral components of passive resistance forces that tend to cause a plan rotation of the bridge. This phenomenon is discussed by Wasserman (8). These forces become too large if the skew is large, and resistance to this rotation causes additional forces in the piles and can overload wingwalls if they are attached to the abutment.

7.6 Wingwalls

Small and compact wingwalls are preferred for integral abutment bridges. Large, skewed wingwalls can generate large forces due to passive earth pressures at a significant lever arm from the abutment. If long wingwalls are necessary, they are best free standing and not connected to the abutment.

8 CONCLUSIONS

There are many successful examples of integral abutment bridges in North America and a lessor number in Australia. This form of bridge construction has been used for steel, concrete and prestressed concrete bridge superstructures over a range bridge lengths and for a limited range of bridge skews.

The performance of integral abutment bridges has been surveyed in the United States and found to be regarded highly by most transportation agencies. Performance has been reported as “good to excellent” with few exceptions. In many cases the long use of integral abutment bridges has been slowly developed and refined to ensure details that can be successfully adopted with known behavioural outcomes. The development of these details is usually based on past performance and refinement, rather than on robust analytical procedures.

There is a reasonably large range of limits adopted by US transportation agencies for integral bridges. These vary because of the wide range of climatic conditions, and because of the individual attitudes of designers facing the design outcomes from imprecise analytical tools when modelling integral abutment behaviour. The typical limits adopted were a maximum unrestrained thermal movement of up to 50mm, maximum bridge lengths of 80m for steel superstructures and 120m for concrete superstructures, and a maximum bridge skew of less than 30 degrees.

Integral abutment bridges offer benefits because there are no expansion joints and few or no bearings to maintain. Greater maintenance benefits accrue to integral abutment bridges in colder climates due to their greater resistance to de-icing salts. In many instances integral abutment bridges also have a lower initial capital cost.

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