Re-evaluating Integral Abutment Bridge Design Practices after Three Decades of Standards in Ontario

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Abstract

In 1993, the Ministry of Transportation in Ontario (MTO) released 'Integral Abutment Bridges', a report which documented 16 bridges constructed with integral abutments in Ontario. The report explained the theory of their design and proposed standard details. This document and the 1996 update which followed, set the stage for over 300 integral abutment bridges (hereafter IABs) built on provincial highways in Ontario since that time. Over the last three decades, extensive research, field trials and international experience has been gained and an update to the guideline is overdue. While standards benefit design engineers and constructors, they can be an impediment to progress and improvement. There is potential to improve detailing and expand the range of superstructure and foundation types which can be incorporated into integral bridges.

This paper reflects on the design theory, the code requirements which have evolved with time and influenced the design details, and the field behaviour in service observed through visual observation and structural monitoring. Despite widespread use, there is still significant variation in the approach taken by design engineers and inconsistency in design assumptions. What is the appropriate earth pressure to assume in design, and how does it compare to actual pressures observed through site measurements? Is frost protection necessary? What types of foundations are appropriate for integral bridges and how flexible do foundations need to be? How should connections be detailed to ensure appropriate force transfer between girders and abutment wall, and between the abutment wall and piles? Working towards an update of the report, this paper reflects on challenges, presents examples which fall outside of the existing standard details, and attempts to answer these questions.

Introduction

The Ministry of Transportation of Ontario (MTO) is responsible for managing the bridges on the provincial highways of Ontario. Assets include approximately 3,000 bridges, 2,000 culverts with spans greater than 3 m, a handful of cut-and-cover tunnels (tunnels exceed 90 m in length), and over 2,000 sign support structures. The MTO maintains a set of standards and specifications for bridge construction, developed in collaboration with industry, which are generally followed by municipalities and transit agencies for the 13,000 other bridges in Ontario (Statistics Canada, 2022).

Structural continuity and jointless details are a well understood preference in bridge design in Ontario. More than half of Ontario's provincial bridges are rigid frames or incorporate integral abutments, semi-integral abutments, integral piers, or link slabs (Lam, et al., 2008). Collectively, the use "jointless details" represents the largest philosophical shift in bridge design in Ontario over the last three decades, eschewing the conventional abutment details ubiquitous in bridges built in the 60s through 80s, and returning to a durability more in keeping with rigid frame bridges pervasive in the 30s to 50s.

The Origins of Integral Bridges

In the 1930s, shortly after the advent of moment distribution which facilitated the analysis of structurally indeterminate structures, reinforced concrete rigid frame bridges proliferated across

Southern Ontario. These were built with one, two or three spans, and typically consisted of solid concrete slabs. Through the 30s and 40s, monolithic connections between the superstructure and the substructure were routine, with the use of bearings reserved for much longer and more important steel bridges. Over the 40s and into the mid-50s, with increasing need for longer spans, solid slab frames gave way to ‘boat frames’, bridges of T-beam cross-section through their mid-span which transitioned to solid sections at the piers, and voided cross-sections.

Rigid frame bridges are distinguished from integral bridges based on the structural behaviour of their abutments. In a rigid frame bridge, imposed displacements at the top of the abutment, due to thermal loads or long-term material changes (shrinkage of concrete), are accommodated by rotation of the relatively stiff abutments about their base. On soil, the footing itself can accommodate that rotation, whereas on non-yielding soil (rock), usually a more explicit ‘hinge’ is designed into the connection between the abutment wall and the footing. Occasionally, these connections are designed as Mesnager hinges, but more often they are reduced sections, with relatively light reinforcing steel across their centroid, thus creating a section of reduced capacity. A Portland Cement Association (1954) publication summarizes the practice of the day. In an integral bridge, in contrast, imposed displacements at the top of the wall are accommodated by means of translation of the abutment wall. The abutment wall is supported on flexible foundation elements, and so long as the abutment is short compared with the spans, the abutment is considerably stiffer than the piles forcing the deformation into the piles, and the piles deform in double-bending to accommodate that imposed displacement. In reality, any bridge with a rigid connection between the superstructure and the abutment wall, will have an abutment which deforms through both rotation and translation. Which behaviour dominates the response is a function of the relative stiffness of the abutment and the superstructure. Fig. 1 shows a single span rigid frame bridge beside an IAB.

Fig. 1. Single span rigid frame bridge (left) and single span integral abutment bridge (right)

In the structural behaviour of a rigid frame bridge, the load from the superstructure is carried, in large part, by bending at the frame corners. The stiffness of the leg and corner of the frame attracts moment and is an essential element to resist the load from the superstructure. In contrast, the corner of an integral bridge does not attract much moment, and the superstructure is usually designed neglecting the resistance offered by any structural continuity between the superstructure and the abutment, at Ultimate Limit States (ULS). Both types of bridges are frames, but the behaviour is influenced by the relative stiffness of the wall and the superstructure. It is possible to build both rigid frame and IABs in either continuous or semi-continuous manner, which affects which load cases cause bending across the frame corner which affects the magnitude of those moments. In this paper, an integral bridge includes any bridge in which live load is resisted predominantly by bending in the superstructure.

An early example of an IAB in Ontario, built in 1962, is shown in Fig. 2. It is a four-span reinforced concrete bridge with rectangular voids, 60.8 m in length. All three piers are monolithic with the superstructure, and each abutment is on a single row of piles with a concrete hinge between the superstructure and abutment wall. Ten similar bridges were constructed along the same section of Hwy 401. They have different foundations, but all have hinged abutment walls. Given the excellent performance and very low maintenance cost of these bridges, it is unfortunate that more were not built with this degree of continuity until the 90s.

Fig. 2. Hespeler Rd Underpass over Hwy 401, an integral abutment bridge in Ontario built 1962

From observation of past bridge designs in Ontario, it is clear that many designers recognized the benefits and opportunities presented by the flexibility of piles in soil. For example, MTO has bridges where each concrete column was supported on a single steel pile. After research and promotion of IABs in the USA in the early 80s (Wolde-Tinsae, et al., 1983), MTO engineers were quickly able to embrace and standardize the approach.

Integral Bridges, Current Practice

Integral Abutment Bridges, Current Policy, and Design Guidance

Since 1993, integral abutments are preferred for new bridges subject to subsurface and geometric limitations. Nearly 10% of MTO's bridges have integral abutments. The current MTO guidance (Husain & Bagnariol, 1996) is the following:

- Types of superstructure to be used with integral abutments are steel and concrete slab on girders
- Bridge length should not exceed 100 m, or not exceed 150 m with rigorous analysis.
- Skews should not exceed 20° (up to 35° with rigorous analysis)
- Abutments should be roughly parallel
- Abutment height should be roughly the same at both abutments and is limited to 6.0 m
- Frost protection should be provided but can be reduced by providing insulation below the abutments
- Abutments should be supported on H-piles
- Wingwall length is limited to 7.0 m, to limit the restraint due to friction between the soil and walls parallel to the movement
- The soil conditions should permit piles of at least 5 m length (with a pre-augered hole or sleeve filled with loose sand for the upper 3 m to increase flexibility if piles are installed through dense or stiff soil)

Integral abutment bridges have a lower initial cost than the same bridge built with conventional abutments, owing to fewer piles, simple formwork, and no permanent bearings at the abutments. The MTO does not have standards for integral abutment post-tensioned concrete bridges but has built only a few post-tensioned concrete bridges in the last two decades. There is an impression that post-tensioned bridges are more costly and take longer to construct than slab-on-girder bridges, although not borne out by the actual cost and schedule when they are built. However, the labour needed on site to construct a post-tensioned concrete bridge is roughly 50% greater than a slab-on-girder bridge for a typical two-span underpass. Institutional knowledge within Structures Office indicates it was intended that integral abutment guidelines be extended to post-tensioned bridges pending further study.

Standard details for an integral abutment include H-piles embedded 600 mm into the abutment stem, each pile confined locally by stirrups as shown in Fig. 3, designed to transfer the full plastic moment capacity of the pile to the abutment stem. Girders are supported on bearing pedestals with neoprene pads for tolerance during construction.

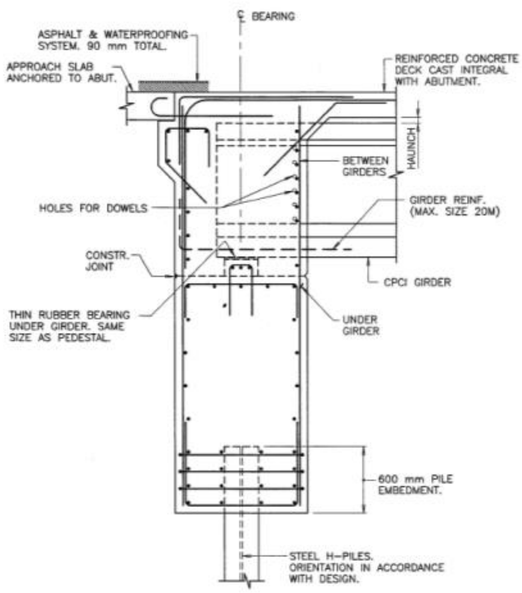

Fig. 3. Typical integral abutment detailing in Ontario (Husain & Bagnariol 1996)

Semi-integral Abutment Bridges

Since 1999, when integral abutments are not feasible, semi-integral abutment details are to be pursued (Husain & Bagnariol, 1999). MTO's standard details for semi-integral abutments consist of a deck end which cantilevers over the ballast wall of the abutment. The cantilevered deck end supports an approach slab which moves as the bridge expands and contracts. Expansion and contraction is accommodated at the end of the approach slab. Semi-integral abutments are more costly than conventional abutments, and usually have joints in the barrier walls which leak over time, causing staining as shown in Fig. 4 and ultimately, deterioration of the wingwalls.

In 2004, the MTO released another guideline to remove expansion joints during rehabilitation by modifying the abutment and deck end to incorporate semi-integral details (Husain, 2004). The existing expansion joint and ballast wall are removed, and the deck is extended to cantilever over the ballast wall and support an approach slab, as shown in Fig. 5. Over 300 bridges have been rehabilitated from conventional to semi-integral abutment details since 2000.

Fig. 4. Typical semi-integral abutments of new steel girder bridges. Note the leakage from the joint in the barrier wall onto the wingwalls.

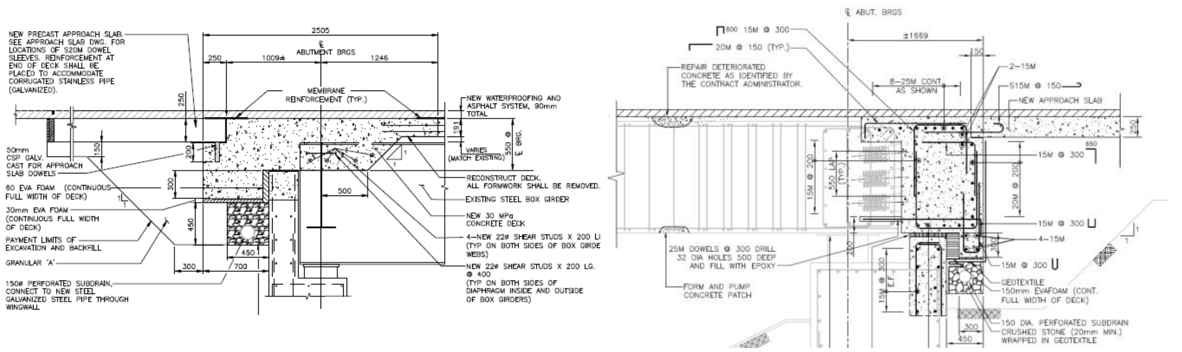

Fig. 5. Examples of bridge rehabilitations including conversion to semi-integral girder ends

Some bridges have been detailed differently with the approach slab extending over the wingwalls as shown in Fig. 6 and Fig. 7. The barriers are cast directly onto the approach slab. With the continuity of the barrier, water is contained within the roadway platform until the end of the approach slab. This detailing also facilitates widening of an existing bridge without extensive modifications to wingwalls, since the approach slab cantilevers over the existing wingwall.

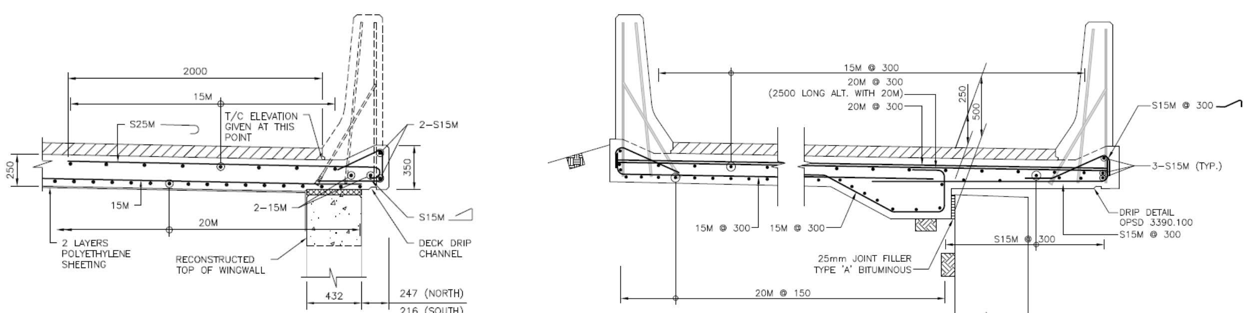

Fig. 6: Semi-integral abutment details with barriers continuous from the deck onto the approach slab with approach slabs sliding over the wingwalls, and expansion joint provided at the end of the approach slab

Fig. 7: Semi-integral abutment details with the barrier continuous from the deck onto the approach slab (overhanging the wingwall in the first image, cast on grade in the second)

Yet another configuration for semi-integral bridges is to have wingwalls supported from the deck end. In this configuration, the full weight of the wingwalls bears on the abutment bearings, and they move with the bridge as they would in an IAB.

Fig. 8: Semi-integral abutment details with wingwalls supported from the deck end

There is a general sense within the MTO Structures Office that, overall, semi-integral bridges are not performing nearly as well as integral bridges. Some engineers have characterized semi-integral abutments as simply shifting deterioration away from the ballast wall to other parts of the structure, namely the wingwalls (leakage, staining and spalling), approach fill (erosion) and sleeper slabs (settlement).

Attempting to quantify these suspicions and improve guidance on semi-integral abutment details, MTO completed a systematic study of 188 semi-integral bridges (79 new bridges and 109 conversions) looking at settlement, cracking and/or leakage. Roughly 7% of the bridges exhibit settlement of the approaches, 30% of the bridges exhibit cracking, and 45% of the bridges exhibit leakage. Leakage tends to increase with time, whereas the cracking is usually related to deficiencies in detailing or construction. Of the various ways of detailing semi-integral bridges, those with wingwalls tied directly into the deck end diaphragms perform the best and they most closely approximate the behaviour of an integral bridge. Bridges where semi-integral deck overhangs sandwiched between wingwalls (as recommended in the guidelines) perform reasonably. Bridges where the approach slab extends over the wingwalls and supports the barriers perform more poorly, exhibiting cracking in half the cases, and leakage in 75% of cases.

Since 2012, MTO has increased its use of corrosion-resistant reinforcing steel, and the current bias towards a semi-integral abutment details over conventional abutments may no longer be warranted. Semi-integral abutments are more costly than conventional abutments, and perhaps that premium is better invested into improving the durability of a conventional abutment. When a conventional abutment is built today, all concrete surfaces exposed to deicing salt runoff from an expansion joint, assumed to be leaking over time, are required to be detailed with corrosion-resistant reinforcing steel. Therefore, it is unlikely that conventional abutments will exhibit the same deterioration which plagued bridges with conventional abutments of the 20th century: deterioration of the ballast wall, deterioration of the abutment wall, and deterioration of the girder ends. Conventional abutments built today are expected to achieve a service life of 75 years without major repair or rehabilitation.

Further improvements to the durability requirements of girder ends and bearings are forthcoming. In a conventional abutment, any runoff reaching the abutment seat should be channelized away (to avoid leakage onto the face of the abutment and staining) and the bearings and girders ends should be protected from runoff, through detailing. Concrete girder ends should be encased, and steel girder ends are coated or detailed with a concrete end wall.

Integral Piers

Integral piers in slab-on-girder bridges, where the ends of the girders are completely encased in a concrete diaphragm, have been built but have not become standard despite lower initial and long-term costs. Integral piers have been most often used in river crossings where a pier is founded on a single line of piles which reduces the size of cofferdam needed to build the pier foundation and allows ice load to be resisted up through the deck, with the entire superstructure acting as a horizontal diaphragm to distribute lateral loads to all substructure locations. In the case of extreme events such as flooding with significant scour, integral pier and abutment connections lead to resilience since any exposed piles are stabilized by the entire structure. As shown in Fig. 9 and Fig. 10, these bridges should be nearly maintenance-free below the deck.


Fig. 9: Steel and precast concrete I-girder bridge with integral abutments and integral piers

Fig. 10: Westminster Drive Underpass over Hwy 401 with integral abutments and integral pier

Integral piers are the modern equivalent of monolithic pier column connections to the superstructure for slab-on-girder bridges. Their appearance is often heavier than when the girders rest on bearings, with the pier cap projecting out from the girder and breaking up the horizontal continuity of the girder, as shown in and Fig. 10. However, when detailed appropriately, they can have both a light appearance and functional advantages, as shown in Fig. 11 and discussed by Sisman and Fu (2004). The load path of integral piers is clear and well-studied (Wassef, et al., 2004).

Fig. 11: Curved composite steel tub girder bridge, and straight precast concrete girder bridge supported on integral pier without bearings

Integral Abutment Bridges, Future Objectives

Rigid frame bridges and integral abutments bridges have exhibited excellent performance in Ontario, with low maintenance costs. Explained in the context of service life design, the low maintenance is attributable to the fact that they have few replaceable components and less severe exposures as compared to an equivalent bridge with conventional abutments supported on bearings.

Given the excellent performance of frame bridges (which includes both rigid frame bridges and IABs), there is value in expanding their range of use. Their use should be expanded to a broader range of structure types and foundation conditions. This requires a broader range of details to support frame bridges, and some existing limitations need to be overcome. An overall goal could be to have every bridge less than 100 m designed as a frame bridge.

Despite widespread use, there is considerable research and testing underway across the world to improve the understanding of these bridges. At the same time, design codes have taken a cautious approach to introducing design requirements for IABs, which has led to some alarming trends in their design, as compared with those built in the time of the initial MTO report in 1993: steel in piles supporting IABs is substantially greater, as is the abutment wall thickness, and the reinforcement. IABs designed today have much stiffer abutments and foundations.

For the efficient design of an integral abutment bridge, the abutment wall needs to be supported on flexible foundations. In a general sense, this flexibility can come from the abutment wall, the piles, and/or the backfill conditions.

Design Challenges and Areas for Improvement of IAB Guidelines

Despite the design guidelines at an engineer’s disposal, the MTO does not have a formal policy or process to lead designers towards a bridge with a high level of continuity and does not mandate IAB design. In recent new highways constructed through public-private partnerships, many bridges were built as IABs, but many were also built with semi-integral or conventional abutments, on high skews with large retained soil system (mechanically stabilized earth) walls. Experience with maintaining past bridges indicates these bridges will have higher than necessary maintenance needs in the 50 to 60 year horizon. Initial cost and long-term performance may sometimes be at odds, but the selection of a particular bridge configuration is often due to institutional policy or code limitations. Less often are truly technical limitations the reason for selection of something other than an IAB, but nonetheless, guidance needs to support the use of IABs over a broader range of scenarios because of the additional effort needed from a designer who departs from approaches that are familiar, or in recent memory of their prior designs.

Types and Geometry of Bridges Suitable for IABs

The 1996 MTO IAB report limits the application of IABs to slab-on-girder bridges. Other jurisdictions do not impose such restriction and there are many examples of cast-in-place concrete IABs, either of reinforced concrete (Skorpen, et al., 2019) or post-tensioned concrete (Kaufmann & Alvarez, 2011) as shown in Fig. 12. There are also plenty of tied arch bridges built with integral abutments (Feldmann, 2010) as well as some trusses. Tied arches and trusses are ideal for IABs owing to the stiffness of their superstructure and correspondingly low imposed rotation at the abutments.

Fig. 12: Spinatobelbrücke, example of a cast-in-place post-tensioned bridge on integral abutments (flexible walls) in Malix, Switzerland (image courtesy of Tiefbauamt Graubünden)

In literature and research, there is far too much emphasis on extending the length of IABs. In MTO's bridge inventory, 86% of bridges have a length of less than 100 m, and 94% of bridges have a length of less than 150 m. Overall, relatively few bridges are beyond the current length limitations suggested for IABs in the 1996 MTO IAB report. By comparison, it is much more important and significant to extend the range of structure types and geotechnical conditions over which IABs are used because those are the reasons that IABs are not selected. Skew is a factor which limits the selection of an IAB. In many of those cases, the trade-off is between a skew bridge with lower initial cost but higher maintenance costs and a square or lesser-skewed bridge which is slightly longer, with correspondingly higher up-front costs but lower long-term maintenance costs. The subsequent sections of this paper propose many strategies to make IABs work with a wide range of conditions.

Long curved bridges can also be designed as IABs, with expansion and contraction due to temperature accommodated by changes in the arc length of the bridge in plan, and a minor rotation about the vertical axis at the abutments. Bearing or column movements due to expansion and contraction of the arch in plan, are therefore transverse to the horizontal profile. This approach is applied to IABs in Switzerland (Kaufmann & Alvarez, 2011) and was applied in Ontario on the Hwy 420 to QEW Fort Erie ramp (Campbell, et al., 1978) in the early 1970s.

Post-tensioned cast-in-place bridges were not eligible for IAB design according to the 1996 MTO IAB report due to concern with long-term unidirectional movements. However, they are indeed feasible with appropriate detailing and commonplace in some geographies. Three cast-in-place post-tensioned IABs have now been designed in-house by MTO engineers. The first is a relatively short two-span solid slab bridge. Imposed displacements on the steel piles are similar in magnitude to those of a longer slab-on-girder bridge, and therefore the deck can be cast monolithically with the abutment walls and then post-tensioned. For the other two bridges, with total lengths of 140 m and 111 m, the deck will be cast on temporary bearings on the abutment walls. After the deck is stressed, the abutment will be jacked against the deck towards the soil, thus preloading the piles to counteract the long-term effects of creep and shrinkage. At that point, the deck end will be placed to encapsulate the anchorages and join the deck and abutment wall. The feedback from the construction of those bridges, starting in 2024, will inform updates to MTO's policy around the design of post-tensioned cast-in-place bridges with integral abutments.

In some cases where rock was relatively shallow, one-sided IABs have been designed as shown in Fig. 13. One side of the bridge has been designed with a rigid frame connection transferring bending of the abutment into the rock mass, while the other side is detailed as an integral abutment on piles.

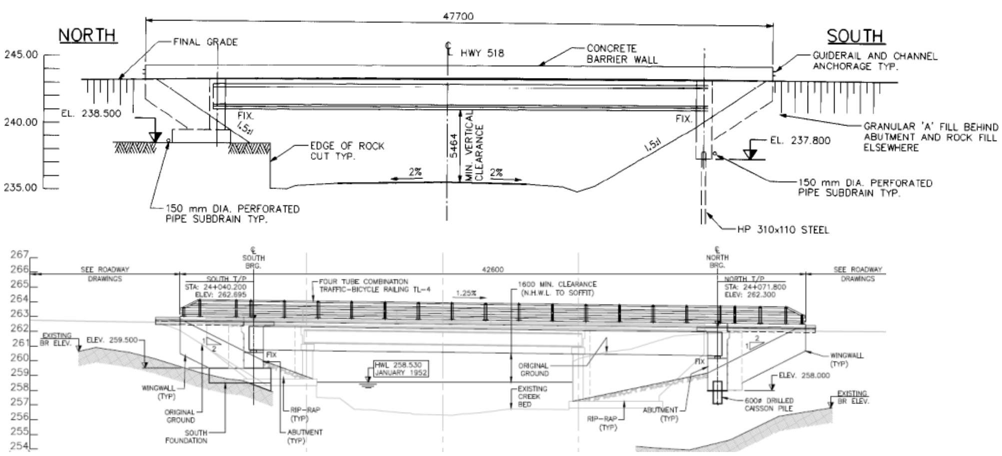

Fig. 13: Asymmetric IABs with one side designed as a rigid frame connection with the bedrock, and the other side designed to accommodate all movement

Abutment Wall Height

For an efficient design of an integral bridge, the abutment wall needs to be supported on flexible foundations. This is best achieved with abutments which are as short as practical, to limit the bending demand introduced from pushing the abutment into the soil.

The 1996 MTO IAB report limits the abutment height to 6 m. While it is a reasonable limit, this height is already taller than optimal for shorter spans, and higher than suggested by most other guidance on this topic. Any increase in height causes an exponential increase in negative moment demand on the frame corner, assuming the piles are sufficiently flexible to deform to imposed displacements, because the horizontal force from the earth is increased due to a larger surface area of the wall, and acts lower down the wall, farther away from the neutral axis of the superstructure. In current designs, it is evident that much of the soil pressure results in flexure of the piles, which drives up the structural demand on them. Where feasible, short abutments should be pursued. Despite the increase in span, short abutments are likely to translate to a more economical structure overall because of reductions in piles and reinforcement at the frame corner. If a taller wall is needed, a more appropriate limit might be a to limit the abutment height to a sixth of the span, and never greater than 6 m.

MTO's practice, in keeping with the 1996 report, has been to detail the abutment with frost depth below the embankment slope. Accordingly, the abutment extends the frost depth, between 1.2 m and up to 2.4 m in Northern Ontario, below the fill line. That additional height of abutment is subject to a horizontal load due to earth pressure as the bridge expands and the abutment pushes into the soil.

Through literature review, it appears that MTO may be the only jurisdiction following this practice. Most jurisdictions require a minimum of 0.6 m embedment of the abutment below the fill line, which is consistent with the minimum requirement of CSA S6-19. While it is often mentioned that this difference in practice may stem from Ontario's cold climate, that is not the case; for example, frost depth is not required for IABs in other cold climates such as Alberta, New York State, Vermont, or Colorado.

Where free-draining soils are used as backfill and placed at the underside of the abutment, and if it is unlikely that the water level rises above the underside of abutment, there is no need to provide embedment of the abutment below the frost depth. Recent designs are progressively moving to shallower embedment of the abutment wall in soil, with insulation provided below and in front of the abutment wall when recommended by the foundations engineer.

An exception to the need for an abutment height limitation arises where abutments are backfilled with lightweight fill consisting of expanded polystyrene (EPS). In that case, since there is no earth pressure on the abutment wall, the wall acts more like a pier and can easily extend to a height of 8 m, as shown in Fig. 14. While it is not appropriate to employ EPS to reduce earth pressures on IABs (for reasons of environmental sustainability), sites where EPS is already required to control settlement of the underlying soils present an opportunity for longer IABs and/or taller abutments.

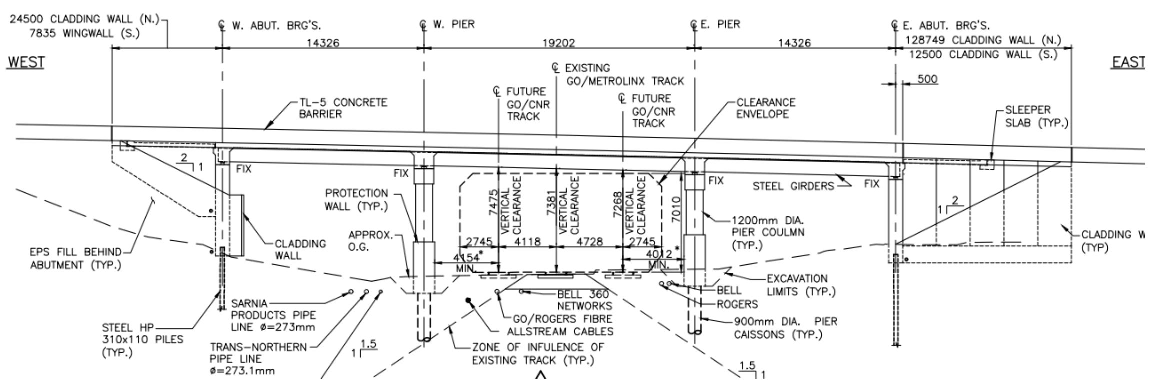

Fig. 14: Hwy 401 eastbound collector Metrolinx Overhead bridge, with lightweight EPS embankments behind 8 m tall integral abutment walls, with a skew of 26°

IAB Pile Design

IABs have traditionally been constructed on H-piles in Ontario, which happen to be the most common type of steel pile in Ontario because of their suitability for the soil conditions. Over the last decade, IABs have also been designed and constructed in Ontario on concrete filled steel tube piles (CFSTs) of 356 mm and 610 mm in diameter, and on 1.2 m diameter concrete caissons.

IABs built today will have two to three times the quantity of steel pile by weight as IABs built three decades prior, as illustrated in the following example. A 73 m long IAB built in 1990, with 5.8 m tall abutments, was supported on HP310x110 piles, oriented in the weak axis, at 1.5 m spacing. In contrast, a 65 m long IAB built in 2020, with 5.5 m tall abutments, has HP360x174 piles oriented in their strong axis, at 1.5 m spacing. Unfortunately, this comparison is not an isolated case but an example of a broader trend.

The piles in the 2020 IAB mentioned above are governed by structural demands at the top of the pile near the abutment. Unfortunately, since one size of pile section is driven for the full length, the additional steel needed only at the top of the piles is provided over the full length of the piles. This is neither cost-effective nor sustainable, and it may be a function of conservatism in design requirements. Respecting the requirements of CSA S6-19, the piles should be designed as beam-columns (commonly interpreted to mean that the support from the surrounding soil should be neglected), a reduction factor should be applied to reduce the resistance of the pile due to driving, and an allowance should be included for loss of thickness of the piles due to corrosion over their service life. When these requirements are combined, the resistance of the pile is reduced significantly, leading to the need for a larger section. One quickly starts chasing their tail towards stiffer piles, thereby attracting greater moments, since most of the bending demand at the top of the pile is due to compatibility. Most H-pile sections are Class 3 or 4 sections in flexure, thereby excluding the benefit of plastic behaviour at ULS.

What is intended by the requirement of S6-19 to design the piles of IABs as beam columns? IAB piles are subject to combined axial compression and bending, and the cross-sectional strength should be satisfied. However, does overall member strength need to be evaluated assuming a free length, or unbraced length, of pile? Full-scale testing on piles driven in stiff clay and in loose soil (compacted fill) found that piles reach their plastic capacity without buckling (Ingram, et al., 2003). Class 3 H-piles are well known to be able to achieve their plastic capacity under axial load when supported by soil. Researchers (Dicleli & Albhaisi, 2003) also affirm that local buckling is the only instability type that need be considered. Vermont, Massachusetts, and Pennsylvania departments of transportation have guidance (VTrans, Integral Abutment Committee, 2009) (Pennsylvania Department of Transportation, 2019) (massDOT, 2020) to designers to design the piles as continuously supported by soil for typical conditions, and check them as unsupported only for a separate analysis after scour takes place, for river crossings where scour is expected.

Piles are installed through soil which provides continuous support to the piles. It is appropriate to design piles as fully braced when they are surrounded by soil. Piles should only be designed as unbraced if they are above a riverbed where scour is expected to expose them, in very weak soils, or if they are completely isolated from the soil. Most importantly, the models used to calculate the demands in the piles should reflect the actual soil conditions surrounding the pile. After scour has occurred around piles, they have a larger unbraced length. This implies a lower axial buckling strength, but the flexural demand is greatly reduced due to increased flexibility over the unbraced length.

A pinned connection between the pile and abutment increases the displacement capacity of the abutment. The 1996 MTO IAB report suggests that the top of the piles can be assumed to be pinned if needed by analysis, but that assumption is not supported by S6-19 requirements and may be overly simplistic. Should piles in an IAB be allowed to exhibit plastic behaviour? Should plastic behaviour be allowed at ULS only or should it be allowed at Serviceability Limits States (SLS) as well? As explained in (Dicleli & Albhaisi, 2004), for a pin to develop at the top of a pile, two conditions must be met. Firstly, the pile needs to undergo large plastic deformations which requires a certain b/t ratio for the flanges (something close to a Class 2 section) so they do not exhibit local buckling prior to yielding. Secondly, thermal-induced low-cycle fatigue needs to be mitigated. Some guidelines (Pennsylvania Department of Transportation, 2019) (massDOT, 2020) and papers (Dicleli & Albhaisi, 2003) (Dicleli & Albhaisi, 2004) propose limits on cyclic curvature in the piles, due to temperature induced cyclic displacements, to mitigate concerns with low-cycle fatigue. The concern is that large cyclic curvatures associated with the swing from summer to winter could lead to fatigue of the pile and eventual failure over time. Daily temperature fluctuations are not a concern. By limiting plastic strains in the pile in service, low-cycle fatigue failure can be avoided within the design service life of the bridge. Pétursson (2015) presents a detailed explanation of low-cycle fatigue, reviews literature and code guidance on this topic, and ultimately recommends that the strain range in the pile be limited to 1% for a 120 year service life, assuming that the bridge is subjected to one large temperature cycle every year. This recommendation is supported by experimental testing on steel pipe piles with D/t of 14.2, and f_y=460 MPa (Pétursson, et al., 2013). In addition to the steel pile design considerations, to support yielding of the steel pile, the abutment stem needs to be designed and detailed to transfer the maximum capacity from the pile at yield. The connection into the abutment, and the abutment itself, should be capacity protected for the pile moment. While it is possible to rely on plastic behaviour of the pile, and the assumption of a pin below the abutment, the designer should evaluate the strains to ensure other failures mechanisms will not arise. Code requirements need to be developed to support plastic behaviour in IAB piles at SLS.

As an alternative to relying on plastic behaviour in the pile for a pinned connection, an actual hinge or pin can be detailed between the pile and the abutment. This has been accomplished by placing the pile within a pocket in the underside of the abutment (Dunker & Liu, 2007), by detailing a rocker, or by reducing the stiffness of the cross-section immediately below the abutment stem, to increase the curvature locally over a chosen length (Feldmann, 2010). A reduction in stiffness can be accomplished by transitioning to a completely different section (e.g. steel rod or vertical steel plate) or by shaping the pile section through cutting out flanges to create dog bone shaped flanges, similar to what is done to create seismic fuses in steel beams in buildings. If a hinged connection is desired, the simplest approach, proven through testing to show stable behaviour and no damage, is wrapping all sides of the embedded pile in 25 mm of polystyrene (Frosch, et al., 2009).

Looking internationally, IABs have been constructed with all types of deep foundations. IABs have been constructed with all types of steel piles: H-piles, tube piles, cruciform sections, and concrete-filled tube piles (Feldmann, 2010). IABs have been built on all types of concrete piles: precast concrete piles, Continuous Flight Augers (CFAs), and concrete drilled shafts/caissons (Dunker & Liu, 2007). There are numerous papers detailing these different systems. Pipe piles present some advantages over H-piles for the design of integral abutment bridges. Firstly, when concrete filled, the surface area exposed to soil is approximately half of that for an H-pile with equivalent diameter, so there is a correspondingly lower loss of resistance due any corrosion over the service life of the structure. Secondly, they have greater I/A resulting in greater buckling capacity at longer unbraced lengths and due to being concrete-filled, the steel wall is assured to reach plastic behaviour under combined axial compression and flexure. Thirdly, they make efficient use of materials and can be more sustainable. In some cases, only the upper section of the pile is filled with concrete, thus increasing the axial compression only closer to the abutment where it is needed. For concrete-filled pipe piles, some sources (Pennsylvania Department of Transportation, 2019) recommend applying a reduction factor of 0.4 to the area of concrete, and to the moment of inertia, when calculating transformed section properties to account for creep.

In summary, it is no longer necessary to restrict deep foundations of IABs to H-piles. In anticipation of changing the guidance around IABs, MTO has designed and constructed several IABs on pipe piles with diameters ranging from 0.3 to 0.6 m, installed by driving or drilling, with and without reinforcement. MTO has also designed IABs on caissons of 1.2 m diameter, with steel casing. Fig. 15 shows two examples of IABs on concrete-filled steel tube piles (CFSTs).

Fig. 15: HSS 356x16 piles supporting the Flagg Road Underpass over Hwy 401 during installation, and the Hwy 101 Ivanhoe River Bridge, supported on three 610 mm diameter concrete filled steel tube piles

Alternative IAB Foundations

For an efficient design of an integral bridge, the abutment wall needs to be supported on a flexible foundation. This is usually best achieved with abutment walls which are as short as practical, supported on a flexible element beneath them, but can also be achieved through the flexibility of the wall itself. Accordingly, it is equally feasible to support IABs on shallow foundations, so long as the spread footing is designed deep enough below the top of the bridge.

In Ontario and internationally, IABs have been constructed on shallow foundations, accomplished in one of three ways. In the first approach, the abutment stem is supported on a flexible column, either steel or reinforced concrete, which is built up from a spread footing. The foundation concept is analogous to older ‘spill-through’ abutments or tall monolithic pier columns on spread footings and creates the same flexible length of column which exists in an IAB on deep foundations, as shown in Fig. 16. In the second approach, the abutment wall itself provides the flexibility needed to accommodate movements, as shown in Fig. 17. In that case, the wall needs to be as flexible as possible but strong enough to resist earth pressure imposed on it. In practice, wall heights of 6 to 10 m, with a wall height to thickness ratio of 10, work well, as shown in Fig. 18. The third approach is to design a short and stiff foundation which slides on the underlying soil (bank pad abutment) as explained in Soubry (2001). The author is not aware of any bridges built with that approach in Ontario.

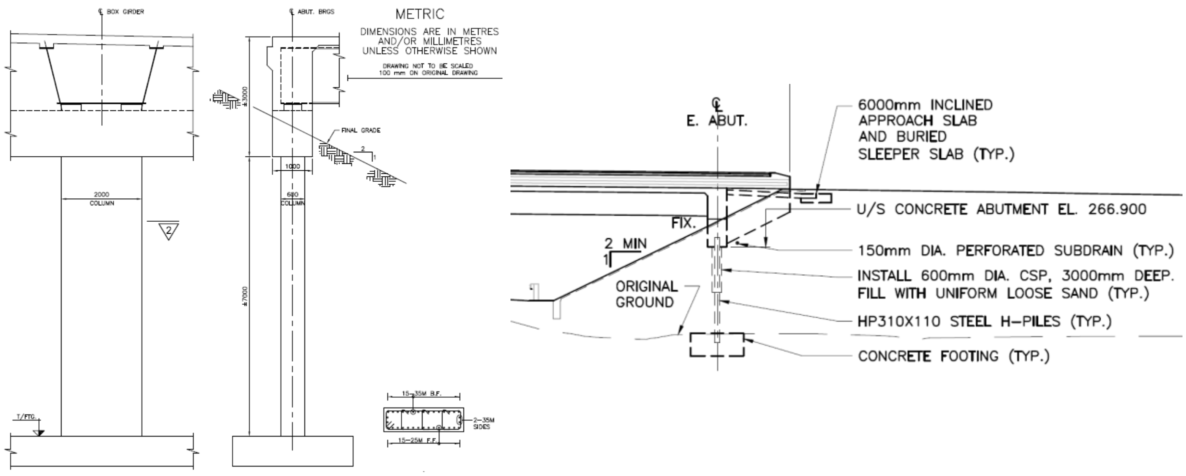

Fig. 16: Examples of integral abutments on shallow foundations with an abutment stem supported on a column built up from the spread footing

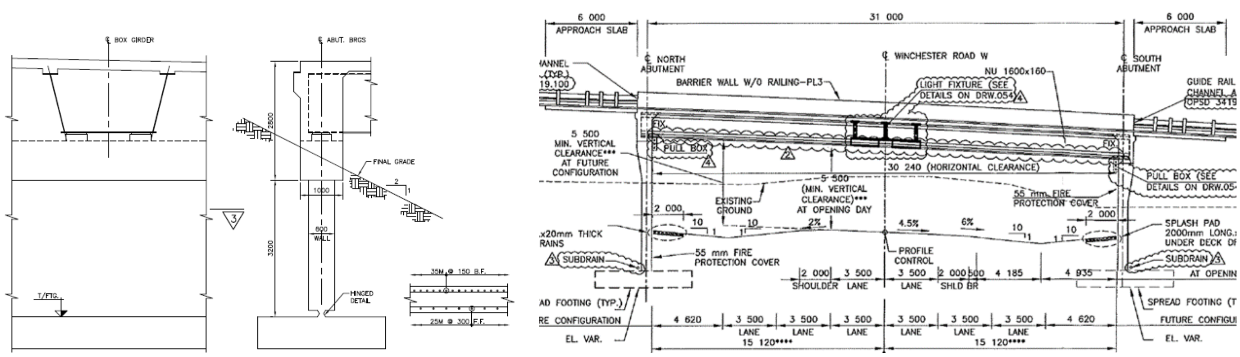

Fig. 17: Examples of integral abutments on shallow foundations where a solid wall provides flexibility above a spread footing

Concrete columns and walls provide a distinct advantage over steel columns for two reasons. Firstly, the concrete creeps. Where columns or wall support a superstructure which is subject to long-term deformations (e.g. creep and shrinkage post-tensioned bridge or shrinkage of a prestressed girder bridge), the moment due to imposed displacement at the top of concrete columns or walls is relieved by creep of those elements in one direction (reduces the magnitude of these moments to roughly a third of the moment from an instantaneously applied displacement) which is very helpful for design. Secondly, concrete columns and walls can be designed to be stiffer transversely than longitudinally. Accordingly, they can support IABs at much higher skews. As with rigid frame bridges, skews of 45° or greater are feasible. In the modeling of these structures, an effective moment of inertia should be used to calculate the forces in the wall, to ensure that the forces due to compatibility (imposed displacement of the superstructure) are not overestimated. Flexible walls and columns are components which can benefit structurally from the use of GFRP reinforcing bars because the effective stiffness of a wall reinforced with GFRP is lower than with reinforcing steel.

Fig. 18: Photos of construction of IABs on shallow foundations where a solid wall provides flexibility above a spread footing. On the example in the right, the wall is partially backfilled prior to casting the frame corner connection between the deck and walls

Connection between piles and the abutment

The 1996 MTO IAB report shows steel piles extending 0.6 m into the abutment, consistent with US guidelines and practice, and schematically shows confinement reinforcement around the pile. In practice, this schematic depiction of confinement has been detailed different ways. Sections of up to HP310x174, oriented about their strong axis, are now used in place of HP310x110 oriented about their weak axis. With an increase in pile section and corresponding increase in bending transferred across this joint, there is a need to examine the confining reinforcing steel around the piles, and for engineers to calculate and detail appropriate confinement in IABs. Strut and tie models can be used to design the load transfer from girder through abutment wall, and into piles (Kalra & Bartlett, 2022). They can be used to design confinement steel around the piles which leads to substantially heavier reinforcement than what is typically detailed in IABs in Ontario.

In field instrumented bridges, it has been observed that the connection between the abutment and top of the pile is not rigid, and some flexibility may be present in the connection due to cracking of the surrounding concrete (LaFave, et al., 2017). Xiao and Chen (2013) investigated failure mechanisms of steel pile to pile cap connections and found that even when designed as pinned connections, and even with shallow embedment, failure is caused by loss of concrete resistance when subjected to pile bending. Frosch et al. (2009) tested several details of piles embedded into abutments, varying the depth of embedment, both unconfined and with confinement reinforcing steel. They were able to prove that an HP310x79 embedded 0.4 m into a concrete section 0.86 m thick, subject to bending about its weak axis, can develop the plastic capacity of the pile at low displacements without any local confinement, but suffered damage of the concrete. Specimens with confinement performed substantially better than those without, exhibiting less damage, greater cyclic deformation capacity, without any loss in their ability to carry axial load. The addition of a spiral around the pile (#4 at 65 mm) was sufficient to capacity protect an HP360x132 bending about its weak axis.

This softening and cracking is a function of the detailing and confinement provided around the pile. It is possible that some IABs currently exhibit such cracking in service, but that cannot be known since these areas cannot be inspected. Confinement should be provided around embedded piles, sufficient to develop their plastic capacity in bending. An updated MTO guideline will tabulate the reinforcement required for various pile sizes.

Connection between girders and abutment

As with the pile to abutment connection, the load transfer between girder and abutment stem is treated empirically following Fig. 3, and may not actually be sufficient to transfer the design moments in some bridges. The practice in Ontario has been to provide reinforcing steel, in the form of L bars, between the deck slab and the back face of the abutment to transfer negative moment at the corner. Little thought is given to how compression is transferred between the bottom flange of the girder and the front face of the abutment, although shear dowels are generally provided through the web at the front face of the abutment. Diagonal cracking observed in the front face of the abutment, below girder flanges, illustrates how the concentrated vertical compression is transferred into the abutment (Husain & Bagnariol, 1999).

The potential deficiencies of this connection become evident when comparing typical Ontario details to international practice (Liang, et al., 2018). Research and testing of these connections (Kim, et al., 2012) indicates that the stiffest load path is the vertical force couple provided between vertical tension at the end of the girder, and compression below the girder flange at the front face of the abutment. Load is transferred in this manner because the depth of the girder and deck (and moment of inertia) is usually greater than the thickness of the abutment wall. The web is the stiffest load path whereas there is a tendency for the deck to be transversely flexible between girders (i.e. when the frame corner is subject to closing moment, the deck between webs does not displace down with the girders and therefore the L bars at between webs are not strained as much as those immediately over the web). Experimental testing indicates that the most effective moment transfer, with the least damage to the abutment and deck concrete, occurs when the girder is extended closer to the back face of the wall, and there is a stiff vertical shear transfer between the girder web and the abutment reinforcement.

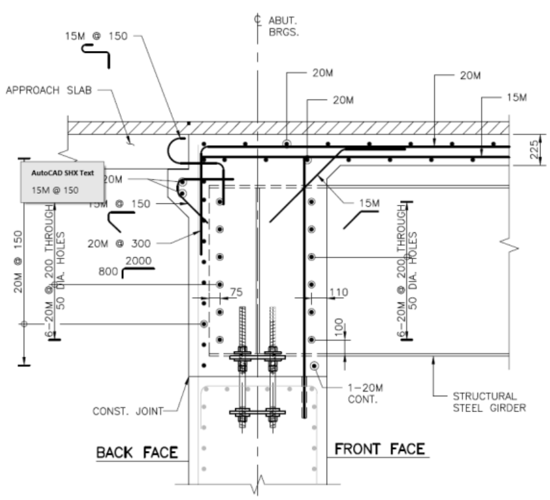

Fig. 19: IAB connection where the girder extends close to the back face of the abutment, and horizontal dowels through the web (similar in behaviour to perfobond shear connectors) transfer the vertical force couple due to bending. Vertical anchor rods used to achieve even bearing in the concrete below the girder flange at the front face, and tolerance/adjustability during erection.

The performance of current details, as presented in the 1996 MTO IAB report, has proven adequate over three decades of service, but detailing of the girder to abutment joint should be improved to improve ductility at ULS and improve performance under extreme events. Fig. 19 shows minor changes to detailing which improve performance.

Abutment Backfill and Earth Pressure

When an IAB expands in the summer months, it pushes into the soil. The magnitude of this displacement is a function of the total length of the structure, and the materials in the superstructure. In design, this behaviour is accounted for by applying an earth pressure to the back face of the abutment wall which lies somewhere between the at-rest and the passive earth pressure. Despite backfilling with well-defined and compacted cohesionless soil, IABs are designed with a relatively crude assessment of the backfill pressures, usually based on figures in CSA S6.1 (Canadian Standards Association, 2021) for sand, which are adapted from NAVFAC. The Canadian Foundation Engineering Manual (CFEM) (Canadian Geotechnical Society, 2007) provides curves for cohesionless soil which are also different. Typically, a triangular earth pressure distribution is assumed, although in some IABs, the displacement at the base of the wall is small, it seems unlikely that a triangular distribution properly reflects the actual loading. For abutments which rotate about their base, BA 42/96 (K* in the Fig. 20) suggests a distribution in which the peak demand exists at mid-height of the wall, and is superimposed on the at-rest earth pressure. Fig. 20 illustrates the difference the assumption of backfill can make on the earth pressure loading, especially for the range of movements of integral abutments.

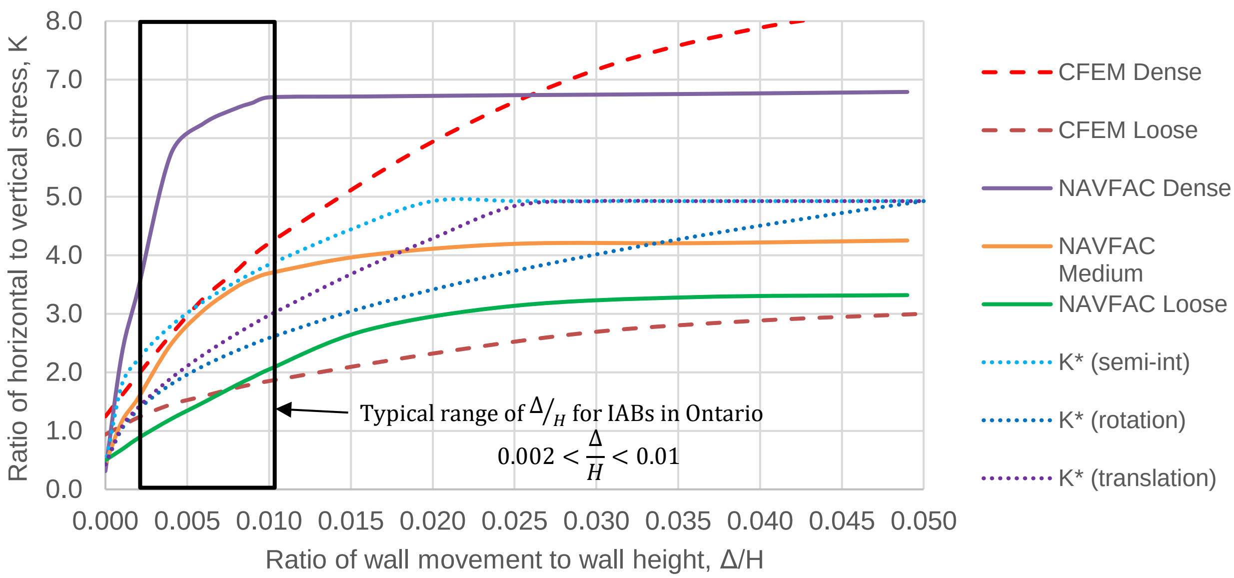

Fig. 20: Passive Earth Pressure Conditions from Various Sources (CFEM Cohesionless, NAVFAC/CHBDC sand, and BA 42/96 UK Standard)

Another matter of debate amongst engineers is whether it is necessary to factor the earth pressures differently at each abutment. Through discussion with many experts, the consensus is that earth pressure at each side of the structure need not be factored differently, unless there is a compelling reason to do so (i.e. different type of fill, different water table). For a bridge on flexible piles, this is justifiable since the fill provides a self-centering effect. If the abutment pushes into the soil more on one side that the other, the entire structure will sway and the earth pressures will equalize on both abutments. As the bridge expands, the system can be visualized as a rigid body with the soil acting as a spring on both sides. Although soil pressure on the abutments is typically modeled as a load, it behaves as both a load and resistance.

For a load which has such a significant impact on the design of the piles and the connection between the superstructure and the abutment wall, there should be a greater certainty associated with the pressure distribution and guidance that is specific to the backfill material used in Ontario, and accounts for the combination of translation and rotation which most integral abutments are subjected to. To this end, the MTO is funding several research projects on this topic which should conclude in 2023, with the hope of developing equations, of a simple form such as those in BA 42/96, for the granular backfill typically used in Ontario. Another approach to improving the accuracy of the inputs is to complete a soil-structure interaction in an FE software such as Plaxis, to determine appropriate earth pressure distribution for the stiffness distribution of the actual bridge's dimensions.

Embankments, Approach slabs, Expansion Joints

Current practice in Ontario for integral and semi-integral abutments is to construct an approach slab at grade, with expansion joints installed at the end of the approach slab for movements greater than 10 mm (bridges greater than 40 m). A sleeper slab anchors the fixed end of the joint as shown at the right side of Fig. 21, and also supports the end of the approach slab. For movements less than 10 mm, the approach slab terminates and the asphalt is saw-cut full depth and filled with hot-poured rubberized compound.

Fig. 21: Expansion joint at the end of an approach slab

The biggest challenge to the use of sleeper slabs at the end of approach slabs is that the embankment fill, behind abutments, invariable settles and the end of the approach slab and sleeper slab settles with it. When the end of the approach slab features only a saw-cut joint, the approach can be repaved to correct the highway profile. With an expansion joint, it is not as simple. One potential solution is to bury the approach slab and distribute the movement over a greater distance of pavement. MTO has studied (Carvajal, et al., 2020), designed and constructed several versions of buried approach slabs, and is carefully monitoring their performance to determine which details should be standardized.

Conclusion

Integral bridges built to date have exhibited good performance after a few decades in service. Other than the settlement of approach embankments, no major problems have been observed to date. Semi-integral bridges have exhibited varied performance. Their performance is not nearly as good as IABs. Integral piers have performed well and appear effective to reduce long-term maintenance costs. They also present opportunities to support bridges which are curved and skewed with respect to the highway or river beneath them, without the need to construct a skewed bridge or wide pier.

Considering their excellent performance, the use of IABs should be extended to more structure types and foundation systems. As demonstrated by IABs built in Ontario, with ‘non-standard’ details and through a scan of international practices, IABs can be built with nearly any type of superstructure and nearly any type of substructure.

New guidance around proportioning and detailing integral abutments in Ontario is needed. Areas which need improved guidance include:

1. Guidance around soil cover to piles supporting integral abutments
2. Pile design requirements
3. Details for integral abutments on spread footings
4. Improved details for the connection between girders and abutment walls
5. Improved details, specifically improved confinement, for the connection between piles and abutment walls
6. Determination of abutment earth pressure, for backfill to abutments used in Ontario
7. Approach slab details to accommodate settlement of the approach embankments.

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