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Sreya Dhar & Kaustubh Dasgupta

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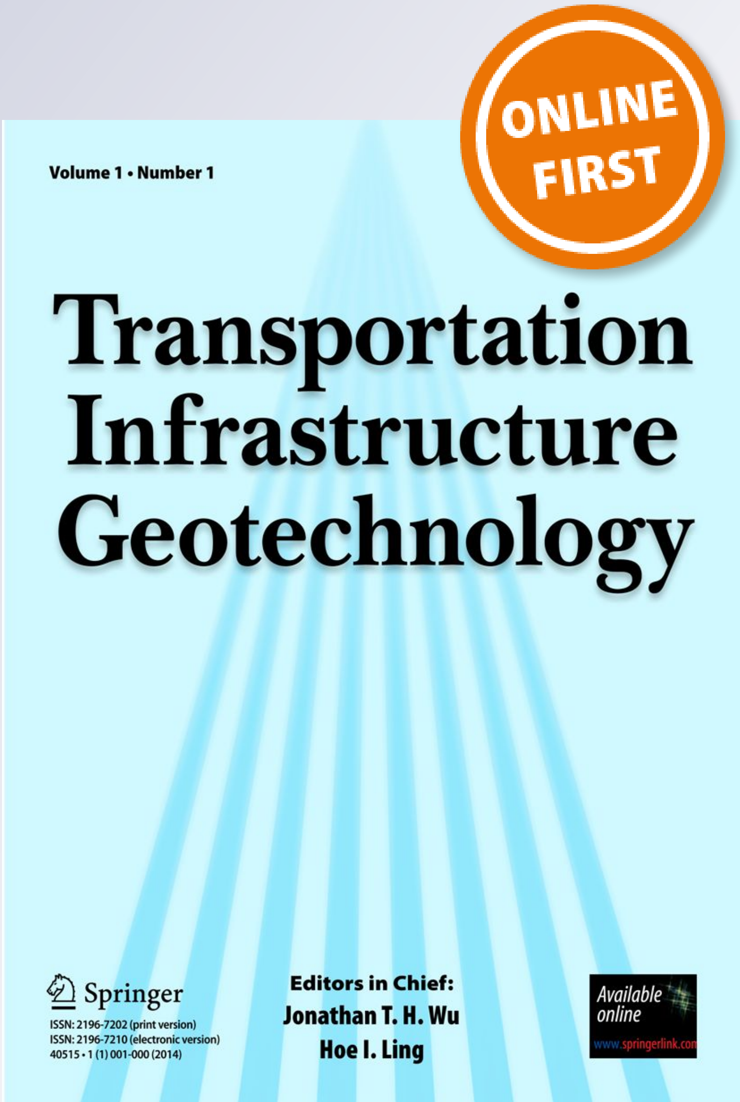

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TECHNICAL PAPER

Seismic Soil Structure Interaction for Integral Abutment Bridges: a Review

Sreya Dhar[1] · **Kaustubh Dasgupta**[1]

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Abstract

In an integral abutment bridge (IAB), the superstructure and the abutment are constructed monolithically at their junction without the presence of any bearing or expansion joint. This leads to a significant reduction in the maintenance cost of the bridge. However, integral connection at deck-abutment junction causes a significant change in the bridge behavior under thermal loading and earthquake shaking as the superstructure (along with bridge deck and girders), abutment, foundation, wingwall, and approach slab may act like a single unit. Different countries and the respective Highway Agencies have adopted different guidelines for design and construction of IABs. Though many advancement in construction of IAB have been made, still there are many aspects which require additional attention. The aim of the present paper is to review the past studies on seismic behavior of IABs performed in the last three decades incorporating seismic soil-structure interaction. A few features are also highlighted which need to be addressed through further studies.

Keywords Integral abutment bridge · Abutment-backfill interaction · Soil-pile interaction

1 Introduction

In conventional bridges, bearings constitute a very important component from the point of functionality. In case of severe earthquake shaking, the bridge deck tends to get unseated at the bearing locations. In integral abutment bridges (IABs) (Fig. 1), the deck-abutment junctions are made monolithic, thus, eliminating the requirement of any bearing (Wasserman and Walker 1996) and also any possibility of unseating of the deck at that location. Analysis of IABs is more complicated as all elements of the bridge are to be considered as a single

✉ Sreya Dhar
sreya.dhar@gmail.com

[1] Department of Civil engineering, IIT Guwahati, Guwahati, Assam, India

system along with soil-structure interaction (SSI). As it is more durable and economical in terms of serviceability and maintenance, it has gained a wide range of popularity over the last few decades. A complete review and up to date information on IAB is still absent in the archive, hence, the authors could not resist the opportunity to brief it in a short technical note. The authors expect that this note can provide a complete outline to the future engineers on introductory knowledge of IAB, covering its importance and applications over past few decades as well as its intricacies that need to be addressed.

Usually, four types of bridge superstructure with integral abutments are constructed, namely (a) steel, (b) reinforced concrete (cast-in-situ), (c) prestressed (precast) concrete, and (d) post-tensioned concrete. Prestressed concrete and steel superstructure integral abutments (Conboy and Stoothoff 2005; Weakley 2005; Maberry and Camp 2005; Lampe and Azizinamini 2000; Arockiasamy et al. 2004) have been constructed extensively in the past; however, the design and construction details have varied from place to place (Maruri and Petro 2005; White et al. 2010). Several modifications have been made in integral abutment and foundation design (Yannotti et al. 2005) which will be discussed in following sections. Currently, three types of jointless construction are being made, namely (a) full integral abutment (FIA) (Fig. 2) (Itani and Peckan 2011), (b) semi integral abutment (SIA) (Fig. 3), and (c) deck extension (Fig. 4) (Weakley 2005; Maberry and Camp 2005; Perkun and Michael 2005). In SIA bridges, there can be the partial force and moment transfer but in FIA bridges, full transfer of moment and forces occurs from deck to abutment. Though in the past, many modifications and improvements have been made to improve the process of behaving like a single unit, a comprehensive review of past studies on IABs incorporating SSI is still absent from archives. The current paper aims to present an overview of the behavioral aspects of IABs involving seismic soil-structure interaction (SSSI).

In this paper, the basic characteristics of an integral bridge are summarized from the past work followed by more complicated behavior of the integral abutment. Then, two of the most important features have been discussed to analyze the bridge under seismic loading i.e. abutment-backfill interaction and soil-pile interaction. At last, a brief review of past numerical and experimental studies is provided to consummate an intuitive overview of IAB.

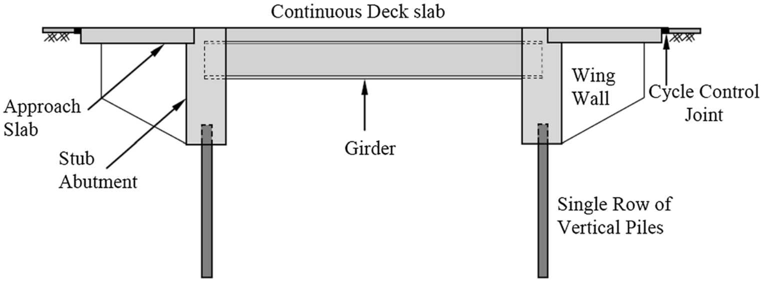

Fig. 1 A typical single span jointless bridge

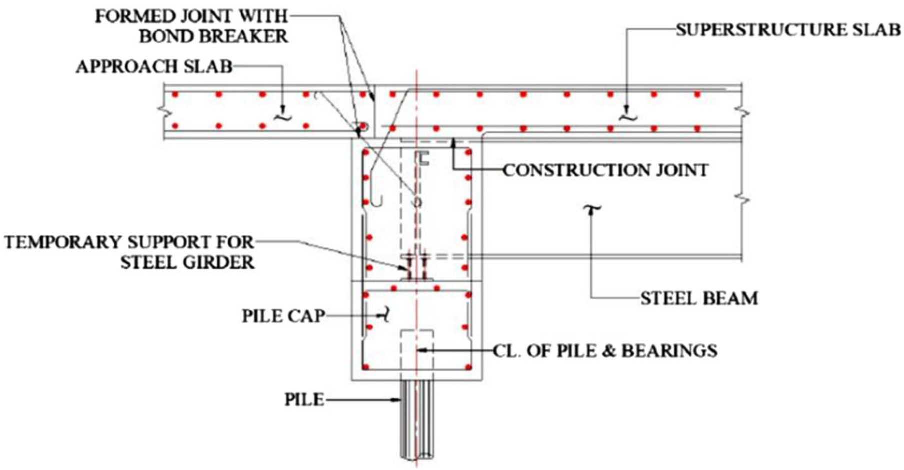

Fig. 2 Fully integral abutment bridge (Yannotti et al. 2005)

2 Basic Characteristics

Before going into the detail, it is very important to be aware of the basic characteristics of IAB, which influence the overall seismic behavior of an integral abutment. There are few features which come under basic characteristics, i.e., length, the skew angle of bridge, and loadings on the bridge. The length of an IAB depends on pile capacity, soil type, and abutment displacement due to temperature variation and seismic excitation (Greimann et al.

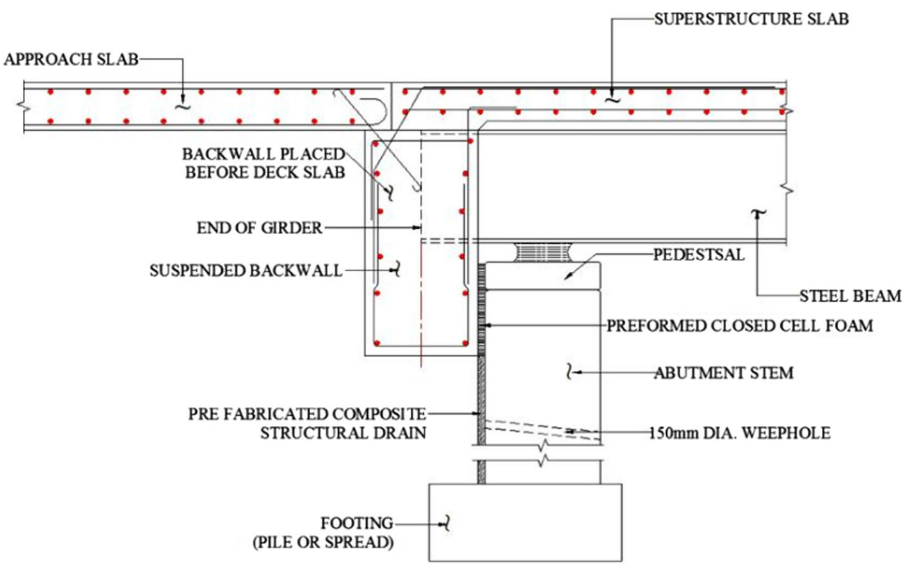

Fig. 3 Semi-integral abutment bridge (Yannotti et al. 2005)

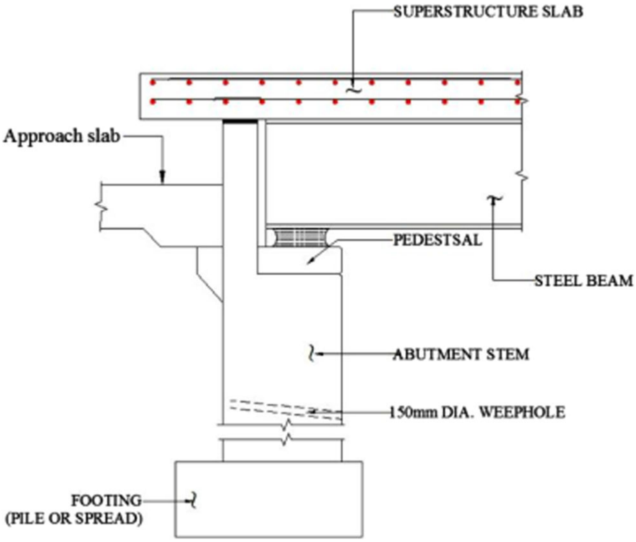

Fig. 4 Deck extension abutment (Yannotti et al. 2005)

1984). Barr et al. (2013) have observed that an increase in span by a factor of two can lead to an increase of bending moment by 60% around the weak axis of the bridge abutment. Although, long span IABs may have total span more than 300 m (e.g., Happy Hollow Creek Bridge with a length of 358 m (Comisu 2005)), currently, researches are continuing in the direction of increasing the length of IABs considering optimization approach for different shapes of pile cross-sections (Lan 2012). However, comprehensive studies involving the prescription of optimal length of IAB based on variation in foundation characteristics incorporating SSI have not been carried out in past studies. The direction of earthquake shaking also affects the behavior of skewed IABs. Bridges with large skew angles may result in a higher level of design forces and moments in different structural components which will further enhance the SSI effect on integral abutment system. The maximum permitted skew angle varies according to the guidelines of different countries and is prescribed as (a) 30° for UK, Finland, and Australia (Gibbens and McManus 2011; White 2007) and (b) 30°–60° in different states of the USA and European countries (Barr et al. 2013; Greimann et al. 1983; Puzey 2012; Quinn and Civjan 2016). In Canada, detailed investigations are required while designing IABs with a skew angle of more than 20°. The same practice is followed in Japan due to the possibility of frequent earthquake shaking. In addition to the primary load effects (dead load and live load), integral bridges are subjected to secondary load effects due to (a) creep and shrinkage, (b) thermal gradients, (c) abutment-backfill Interaction (ABI), and (d) soil-pile interaction (SPI) (Arockiasamy et al. 2004). For the sake of the main focus of the paper, the discussion on loadings is limited within ABI and SPI in the following sections. The basic characteristics can be used as key parameters to assess the effect of SSSI under strong earthquake shaking.

3 Abutment Behavior

The total earth pressure on the abutment during an earthquake is contributed by three components, namely (1) the static pressure due to gravity loads, (2) lateral pressure induced due to the displacement of the wall towards the backfill from inertial loading, and (3) earthquake-induced lateral pressures due to ABI (Matthewson et al. 1980). For the analysis of an integral abutment-foundation system, three different stiffness contributions are considered, namely (a) longitudinal or translational stiffness of abutment, (b) rotational stiffness of the abutment wall-backfill system, and (c) stiffness of underlying foundation. By lumping various components of stiffness at different locations in the structural model (Fig. 5), abutment stiffnesses can be derived (Petursson and Kerokoski 2011). Active earth pressure tends to get mobilized at a very small lateral displacement when the abutment moves away from the backfill (Barker et al. 1991). Most of the countries in Europe and the American States prefer to consider full passive pressure behind the abutment. During earthquake shaking, inertia forces generated at deck are transmitted to the backfill soil through integral abutments and to the surrounding soil through the foundation. The deck-abutment joints are thus subjected to high stresses under seismic loading. The past studies on the behavior of integral abutment and its foundation system have been discussed in the following sections.

3.1 Abutment Foundation

Usually, fixed head steel H-piles are very widely used for abutment foundation in IABs. Other than H-piles, different types of abutment foundation (Dunker and Liu 2007) include steel pipe piles, precast concrete piles, timber piles (Kamel et al. 1996),sheet piles (England et al. 2000), spread footing and X-shaped (cross-shaped) piles (White

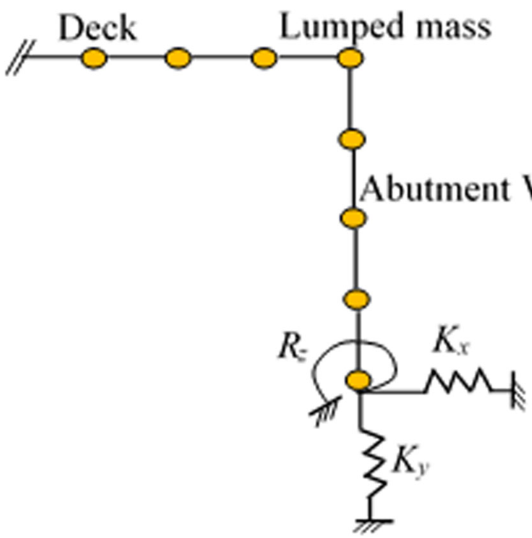

Fig. 5 Lumped mass and stiffness at different locations of the structural model (where, R_z, K_x, and K_y are rotational, horizontal, and vertical stiffnesses, respectively)

et al. 2010). The spread footing is generally avoided for multispan IABs due to differential settlement. Use of batter piles for skewed IABs is also not suggested in construction practice (Hassiotis et al. 2006).

Pile to pile-cap and pile to abutment connections are mainly based on two types of connections, namely fixed and hinged. Hinged pile head can be constructed by anchorage or providing carpet wrapping at top of theabutment piles. When an IAB is supported by flexible or hinged pile heads, all longitudinal forces are taken up by abutment-backfill, bridge deck, and to some extent by the flexible abutment piles during earthquake shaking. A pinned abutment-pile head connection significantly increases the displacement of the bridge due to increased translation and rotation of the abutment (Dicleli and Albhaisi 2003) but it reduces the force transferred from abutments to piles significantly (Arsoy et al. 2002). Generally, steel H-pile is oriented for weak axis bending is the case of hinge connections. Concrete piles are not recommended because under lateral loads cracks may form in tension and significantly reduce the axial load carrying capacity of these piles. Just by providing carpet wrap at the abutment-pile head junction is unable to produce full hinge connection at the abutment-pile head joint (Abendroth et al. 2007). As prescribed by the Virginia Transportation Research Council (VTRC), a hinge connection of an integral abutment to pile cap joint is shown in Fig. 6.

Usually, abutments are supported by a single row of piles along longitudinal direction through weak axis bending (Wasserman and Walker 1996; Arockiasamy et al. 2004; Quinn and Civjan 2016; Arsoy et al. 2002; Burke 1993; Nielsen and Schmeckpeper 2001; BSDC 2017) to accommodate higher flexibility. This results in a desirable behavior for the abutment-pile system with avoidance of concrete cracking even for skewed bridges and bridges with H-piles under seismic loading. However, it also tends to increase the stresses in piles leading to the possibility of plastic hinge formation at pile head under seismic excitation. Although H-piles are not suggested to undergo strong axis bending (Abendroth and Greimann 2005), the combination of

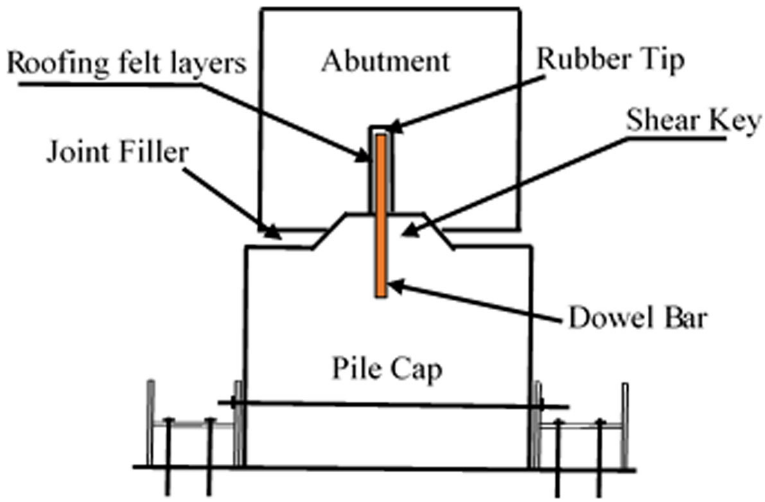

Fig. 6 Schematic diagram of an integral abutment to the pile-cap joint as hinge connection (cross-section) (Arsoy et al. 2002)

strong axis bending and stiff foundation soil reduces stresses in piles and increase forces in concrete deck girders (Huang et al. 2008). So, pile design and orientation should vary with specific bridge situation. The design and orientation of abutment piles should be based on specific bridge characteristics, temperature variation, and type of abutment-backfill.

3.2 Backfill, Passive Pressure, and Lateral Loading

Most of the American states prefer well-compacted granular materials as backfill. Earthquake-induced lateral loading can mobilize passive soil pressure resistance through sliding or horizontal displacement of abutment towards backfill. For walls of low heights (up to 1.5 m), it is expected that the inertial effect under earthquake shaking should be small, thus passive resistance is computed using static pressure distribution. The Mononobe Okabe (MO) method of determining seismic passive pressure coefficients for abutments is not recommended due to its various limitations (SCDOT 2010). Considering wall friction and soil surface failure, seismic passive earth pressure coefficients have been suggested in past studies (Shamsabadi 2006; Anderson et al. 2008; Shamsabadi et al. 2007). When superstructure inertia forces are transmitted into the backfill through abutment, adequate passive resistance must be available to avoid translation and rotation of the abutment. It is recommended that abutments should be designed to restrict lateral displacements up to approximately 0.091 m in order to avoid severe failure under dynamic loading (AASHTO 2002). Damping arising from ABI is also observed to be significant in several experimental studies for relatively short IABs (up to 60 m length) (Douglas and Reid 1982). Based on experimental and analytical data (Hassiotis et al. 2006; BA 42/96 Amendment No. 1 2003), a seasonal movement of 0.020 m can be allowed for short span IABs.

The variation of the backfill pressure as a function of the abutment displacement towards the backfill has been obtained from past studies (Barker et al. 1991; Claugh and Duncan 1991). Ting and Faraji (Ting and Faraji 1998) compared the design curves are given in Canadian Foundation Engineering Manual (CFEM) (CFEM 2006) and NCHRP (Barker et al. 1991) with experimental results for translation and rotation of the abutment wall. For dense soil, design curves given in CFEM are recommended for ABI while for loose and medium soils, the curves given in NCHRP are suitable. Another study has prescribed equations to calculate the coefficient of passive earth pressure which suits the NCHRP design curves (Bonczar et al. 2005). Acceptable comparisons are achieved from design curves for dense and medium soils given in CFEM and NCHRP with the suggested expressions from past researchers for modeling of integral abutment (Kumar 2008). Caltrans (Caltrans 2004) has recommended the estimation of seismic passive soil resistance behind bridge abutments based on full-scale experimental results. An approximate quasi-linear relationship between seismic passive earth pressure and wall movement has also been established (Dicleli and Albhaisi 2004; Arsoy et al. 1999). "Log-spiral" hyperbolic load deflection (HLD) curve (in Fig. 7) for seismic passive pressure has been proposed to represent the nonlinear ABI for monotonic pushover analysis (Shamsabadi et al. 2007; Duncan and Mokwa 2001; API RP2A-WSD 2000). Later from extended hyperbolic force-deformation (HFD) curves (Shamsabadi et al. 2010), generalized HFD curves are obtained for passive backfill resistance (Khalili-Tehrani et al. 2016). Extended HFD and generalized HLD curves are

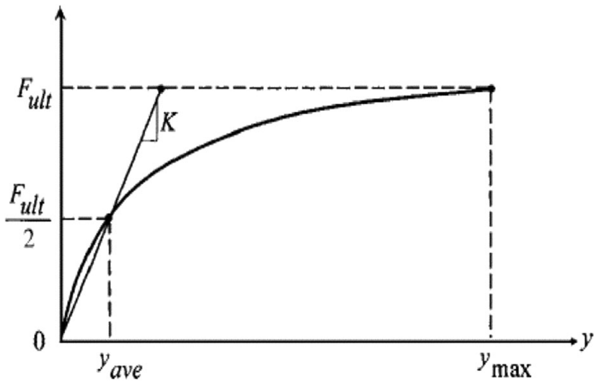

Fig. 7 A hyperbolic force-displacement formulation for abutment-backfill interaction (where, F_{ult} is the maximum abutment force (kips) for the entire wall; K is secant stiffness; y_{max} and y_{ave} are maximum and average displacement (in inches), respectively) (Shamsabadi et al. 2007)

defined at the deck-abutment junction. A direct displacement-based design was introduced by Mitoulis et al. (Mitoulis et al. 2015) to evaluate the stiffness and damping properties of the deck-abutments-backfill system under dynamic loading.

Researches have produced relatively simple and cost-effective design solutions for reduction of irreversible movements of IAB as well as strain ratcheting of backfill soil (Hassiotis et al. 2005). Additive seismic lateral earth pressures on abutments can be reduced by using a variety of modern design approaches, for example (a) using geosynthetic reinforcement or geogrid in backfill soil (Zornberg 2007; Tatsuoka et al. 2014; Argyroudis et al. 2016), (b) using rubber-soil mixture or reused tyre aggregates in backfill (Argyroudis et al. 2016; Mitoulis et al. 2016; Mitoulis 2016), (c) installing buried approach slab (Wendner and Strauss 2013), (d) using polyethylene sheets below approach slab to reduce friction with backfill soil (Mistry 2005), (e) by different ground-improvement procedures (Horvath 2005, 2000), and (f) by retrofitting the jointed bridges to IABs (Jayaraman et al. 2001). Use of rubberish material with backfill soil reduces abutment top displacement, “ratcheting effect” and the gap between backfill and abutment. Due to irregular stress concentration, small bumps at the end of the bridge may occur. This additional strain can also be reduced adding rubber chips in backfill material.

4 Soil-Pile Interaction

In IABs, the forces coming from primary loads (i.e., dead and live loads) and secondary loads (i.e., creep, shrinkage, static, and cyclic loads) are resisted by ABI and SPI. Apart from ABI, SPI plays a major role in the load transfer mechanism of SSI. Piles in IAB are designed to carry forces coming from substructure and superstructure. Also, the piles should be flexible enough to accommodate lateral movements without failure, irrespective of short or long piles. Basic assumptions of 3D spring-dashpots to model SPI were initially proposed by Greimann et al. (1986). Lateral displacement of the pile significantly affects the capacity of the pile to transfer the load to the ground in the lateral direction. However, lateral displacement does not affect the end bearing resistance of flexible piles. Maximum lateral displacement of the pile head below which the frictional resistance is assumed to be unaffected have been established as 2% of the pile diameter (Fleming et al. 1992). The ultimate pile capacity tends to decrease with an increase in horizontal displacement at the pile head (Girton et al. 1991). The ultimate displacement capacity is reached when either the plastic hinge is formed in piles or the abutment fails in shear or flexure. In addition, these assumptions generally do not take into account the interaction between piles to bedrock. A design procedure is proposed where the depth of bedrock is also considered to evaluate the effective length of piles (DeLano 2002). Pile bearing resistance should be verified in the field during pile installation using static load tests and dynamic tests (AASHTO LRFD 2012). Plastic design of steel H-piles for IAB has also been suggested by load factor design (LFD) method (Huckabee 2005).

Soil characteristics in the SPI can be described by three types of force-displacement curves in piles (API RP2A-WSD 2000), namely, (a) variation of horizontal lateral force (p) with lateral displacement (y), (b) variation of vertical skin friction (f) with vertical displacement (z), and (c) variation of end bearing capacity (q) with pile tip displacement (t). These curves have been used widely in the analytical models under dynamic loading. Axial and lateral force effects have been considered uncoupled and the concept was first developed by McClelland and Focht (1958). Based on extensive field tests on piles, Matlock (1970) has proposed *p-y* curves for piles in soft clay. Later, Reese et al. (1975) have proposed a family of *p-y* curves for stiff clay and sand based on experimental field testing of piles. Afterward, a unified curve for both soft and stiff clays has been defined by Sullivan et al. (1979). Several other methods like Hansen's theory (Hansen 1963) and "Broms" method (1964) have also been proposed for estimation of lateral pile resistance for the same soil conditions (Fan and Long 2005; Shia 2005). The most commonly used API method (API RP2A-WSD 2000) has been implemented for soft clay (Matlock 1970), stiff clay (Reese et al. 1975), and sand (Murchison 1983) under cyclic loading.

A thorough discussion for the determination of *p-y* curves is given by Wang and Reese (1993). COM624P software can be used for the analysis of SPI (Wasserman and Walker 1996; Wasserman 2001) and has been approved by a study at the University of Tennessee (Burdette et al. 2004). The unbraced length of the pile is determined from the identification of the points of zero moments at several depths of the pile and the longest of these distances is used in subsequent nonlinear calculations as "effective length." Previously, field tests have been performed on driven piles to compare the observed ultimate strength with that of the computed strength, using AASHTO (1996) and AISC (1996) column design equations. The results from the field study have shown that the prescribed equations predicted overly conservative values for the ultimate lateral bearing capacity of the piles (Ingram et al. 2003) because considering piles as unsupported between inflection points does not take into consideration the influence of lateral resistance from the surrounding soil. So, proper measures should be taken during lateral load design of different types of concrete piles which are partially included in the current code version (AASHTO LRFD 2012).

The wingwalls are potentially capable to increase seismic demand on the integral abutment system. The abutment and wingwall may not behave as a rigid block even if abutment is supported on flexible piles (Mourad and Tabsh 1998). Because reducing the number of piles under the abutment greatly affects the axial load on piles but it will not significantly affect the tension force in piles under the wingwalls. The axial stresses in the piles are not affected by the type of wingwall and fixed or hinged abutment-pile joint. In the USA, guidelines recommend not to build piles under wingwall as it would restrict the rotation of the integral abutment system which may further induce additional forces on the structure (White 2008). The level of compaction behind the abutment affects the overall response of the IAB. Generally, the axial forces and bending moments in the deck increase and the peak moments in the piles decrease when the backfill soil is varied from loose to dense but for H-piles moment increases once the foundation soil changes from loose to dense (Faraji et al. 2001; Zhao et al. 2011). Very little work concerning the capabilities of integral abutments founded on short piles has been carried out. Existing design procedures preclude the use of piles below effective length because of the assumptions which are generally based on or validated for longer piles only (DeLano 2002). Stress on the piles may increase due to several reasons like long bridge span, stiff soil, deep girders and weak axis bending. The spatial extent of the bridge-foundation system is large which necessitates an appropriate finite element mesh to provide adequate modeling resolution. So, pre-processing and output visualization in 3D finite element analysis (FEA) of the bridge-ground system can be quite tedious and time-consuming (Elgamal et al. 2008).

Beam on Dynamic Winkler Foundation model (BDWF) (Makris and Gazetas 1992) has been established for SPI which states that each subdivided layer of soil should be represented by a series of independent, discrete springs in the vertical and horizontal directions. The soil surrounding the pile is considered to be divided into two zones, namely (a) near field and (b) far field. Strong material nonlinearity is expected in the near field zone while soil behavior would be primarily linear elastic in the far-field zone. If viscous spring-dashpots are modeled in a parallel configuration with the nonlinear spring-dashpots (hysteretic soil in near-field) for capturing the effects of radiation damping, it is referred as parallel radiation damping of the soil-modeling method (Kagawa 1980a; Kagawa and Kraft Jr. 1980b; Badoni and Makris 1996). If nonlinear hysteretic spring-dashpot is placed in series with the linear viscoelastic spring-dashpot (radiation damping), it is termed as series radiation damping of soil modeling (Novak and Sheta 1980). Series radiation damping method is more realistic as compared to the parallel radiation damping method as it avoids the possibility of unrealistically large damping forces in the near field zone (Wang et al. 1998). Radiation damping in far-field soil domain of bridges occurred due to energy dissipation of waves radiating out into soil away from the bridge foundation (Gazetas and Dobry 1984).

4.1 Computational Approach

Many finite element software packages have been developed to analyze the two dimensional (2D) or three dimensional (3D) SPI like OpenSeesPL (Lu et al. 2011), IAB2D, IAB2D (McBride 2005), ANSYS (2013), CSiBRIDGE (CSI 2016), OpenSees (Mazzoni et al. 2007), Abaqus (Hibbitt, Karlsson, and Sorensen, Inc 2014), ADINA (2017), MIDAS (MIDAS Civil 2017), and LUSAS (2014) which can incorporate SPI in different ways. Proper parametric values incorporated in the numerical programs can produce results close to those obtained from the experimental tests (Greimann et al. 1986). The near or far field soil can be modeled as a linear or a nonlinear spring-dashpot element in series or parallel configuration to each other. To this end, continuous soil domain can be modeled for analytical purposes with realistic soil properties (Elgamal et al. 2008; Zhang et al. 2008; Dhar et al. 2016). Different types of geotechnical domain modeling for dynamic analysis with proper boundary conditions have been illustrated in Kontoe (2006).

In the case of 2D modeling method, continuum soil domain is generally modeled with quadrilateral or triangular finite element mesh with suitable constitutive properties of soil. FEA indicated that SIA offers benefits over FIA, such as reducing the pile stresses, especially during contraction of the bridge. In addition, the interaction between the approach fill and the foundation soil creates favorable conditions with pile stresses (Duncan and Arsoy 2003). FE programs also account for the gap formation at the soil-pile and abutment-pile interfaces by introducing gap element or contact element which can capture the realistic behavior of an integral bridge under dynamic loading. Zero-thickness interface elements can be used to model soil-pile and abutment-backfill interactions to allow slip at those interfaces (Zhang et al. 2008). Modeling of contact or interface elements has been discussed in past studies (Hibbitt, Karlsson, and Sorensen, Inc 2014; Zhang et al. 2008; Kolay 2009; Gentela and Dasgupta 2012). Discrete element method (DEM) can be used for numerical analysis to model interaction between different rubber-soil mixtures and abutment (Cui and Mitoulis 2015). Three dimensional (3D) finite element models of IAB have increased complexity as well as computational requirements. However, unlike 2D models, 3D models can account for skew effects (Deng et al. 2015) and the effects of eccentric loading. 3D soil continuum can be modeled by 8- or 20-noded brick element in FE software.

5 Field Studies

IABs equipped with instrumentation provide valuable insight into the behavior of integral piles due to loading from traffic, earth pressure forces and temperature changes. Recently in several countries, short span old conventional bridges are updated to IABs for better performance. Measures are taken for IABs to relieve the earth pressure on the piles and for abutments to permit sufficient longitudinal movements. The void space created behind the abutment can be filled with corrugated metal supported by specifically thickened pressure relief compressible strips (Jorgenson 1983). The piles which are rigidly connected to the bottom of the abutment will bend in double curvature. To eliminate the bending action, integral abutment and pile connection can be provided as hinged.

In a fully integral bridge, internal piers should be capable of lateral movements, if not, then some base isolation system should be arranged in between superstructure and internal piers (Frosch et al. 2005). SIA can be constructed as hinged at the top and bottom of abutment by neoprene bearing strip transferring the force by padded dowels (Arsoy et al. 2002) and should be used where rigid soil and long-span bridges are necessary (Burke Jr 2009). Recently, in many countries the debonded link slab system is introduced at intermediate pier joint to accommodate bridge deck deformation during cyclic loading (Connal 2004; Saber and Aleti 2012; Aktan et al. 2008). Highway overpasses less than 60-m length are suitable for construction as IABs (Torricelli et al. 2012). In New Zealand, FIA and SIA are permitted if the rotational component is taken into consideration in analysis for bridges more than 60 m length (Jamieson 2009).

Experimental results (Springman et al. 1996) suggest that loosely placed backfill should not be used, regardless of whether or not an approach slab is used; although loose backfill reduces the force coming on the bridge during dynamic loading. To prevent settlement of approach slab, US guidelines recommend the use of drag plates or buried approach slab with the special repairable connection between slab and abutment (Frangi et al. 2011). The strain in approach slabs does not depend on whether the approach slab is a precast or cast-in-situ type (Phares et al. 2013). Sometimes, the measured earth pressures on the bridge abutments may be quite high because of well-compacted backfill (Kerokoski and Laaksonen 2005). Other than proper backfill compaction, dynamic response of the bridge structure is also governed by geotechnical characteristics of soil strata. Due to the nonlinear inelastic response of underlying soil during earthquake shaking, large soil deformation imposes large residual displacements and seismic forces on the IAB after strong earthquakes (Elgamal et al. 2008; Zhang et al. 2008). A special type of steel-concrete composite joint is proposed for IABs by Briseghella and Zordan (2015) which is easy to construct and gives satisfactory response under cyclic loading. Field studies have been carried out with analytical verifications for skewed bridges to understand detailed behavior of skewed IABs (Deng et al. 2015; Wright et al. 2015; Olson et al. 2013; Quinn 2016). Based on comparisons of the finite element models with field studies, it can be concluded that finite element models do provide excellent matching to approximate the actual behavior of an IAB.

6 Summary

The SSSI plays a very important role in the behavior of IABs. In this study, the authors have attempted to review the relevant past researches and from those, the salient findings are documented. This technical note has been written to provide a brief summary of the past researches which have been accomplished and which are yet to be achieved. Authors believe that this paper would prepare the practicing engineers and researchers regarding the difficulties of bridge mechanism and construction procedures. Some of the important features of IAB from the past literatures are summarized in Table 1. From the past reviewed works, guidelines for the seismic behavior of IABs on steel piles are prescribed, but design recommendation on RC piles of IABs has not been commented in any design guidelines. Thus, experimental and computational research needs to be focused in that direction. Use of geosynthetic materials as backfill is yet to be implemented in many countries. The salient aspects from the review of the past studies are summarized below.

- Highway overpasses and short span footbridges (up to 60 m span) should be constructed as IABs and can be retrofitted from jointed bridges to jointless bridges easily. Most of the countries (except high earthquake-prone countries like Japan) permit a skew angle of 30° or more, which have not been found to be hazardous for the bridge under dynamic shaking. Again, geotechnical characteristics of the foundation soil are required to be investigated in seismic analysis of the bridge.
- Series radiation damping method for BDWF is more desirable than the parallel radiation damping of soil modeling. Based on comparisons of the finite element models with field experiments, it can be concluded that finite elements models do

Table 1 Important features of IABs from past literature reviewed works

Reference no.	Important features
(Wasserman and Walker 1996; Arockiasamy et al. 2004; Quinn and Civjan 2016; Arsoy et al. 2002; Burke 1993; Nielsen and Schmeckpeper 2001; BSDC 2017)	Generally, integral abutments are supported by a single row of piles along longitudinal direction through weak axis bending to accommodate higher flexibility.
(Lan 2012)	Researches are continuing in the direction of increasing the length of IABs considering optimization approach for different shapes of pile cross-sections.
(Gibbens and McManus 2011; White 2007; Greimann et al. 1983; Puzey 2012; Quinn and Civjan 2016)	The maximum permitted skew angle varies from 30 to 60 from the different region all over the world; exception: no permitted skew angle at Japan and Germany.
(Hassiotis et al. 2006)	Use of batter piles for skewed IABs is also not suggested in construction practice.
(Arsoy et al. 2002)	A pinned abutment-pile head connection significantly reduces the force transferred from abutments to piles.
(Douglas and Reid 1982)	Damping arising from ABI is also observed to be significant in several experimental studies for relatively short IABs (up, 60 m length).
(Barker et al. 1991; Ting and Faraji 1998; CFEM 2006; Bonczar et al. 2005; Kumar 2008; Caltrans 2004; Dicleli and Albhaisi 2004; Arsoy et al. 1999; Duncan and Mokwa 2001; API RP2A-WSD 2000; Shamsabadi et al. 2010; Khalili-Tehrani et al. 2016)	Designed hyperbolic nonlinear force-displacement curves for ABI for different types of backfill soil have been prescribed.
(Hassiotis et al. 2005; Zornberg 2007; Tatsuoka et al. 2014; Argyroudis et al. 2016; Mitoulis et al. 2016; Mitoulis 2016; Wendner and Strauss 2013; Mistry 2005; Horvath 2005; Horvath 2000; Jayaraman et al. 2001; Jorgenson 1983)	Additive seismic lateral earth pressures on abutments can be reduced by using a variety of modern design approaches.
(Fleming et al. 1992)	Maximum lateral displacement of the pile head below which the frictional resistance is assumed to be unaffected have been established as 2% of the pile diameter.
(McClelland and Focht 1958; Matlock 1970; Reese et al. 1975; Sullivan et al. 1979; Hansen 1963; Broms 1964; Fan and Long 2005; Shia 2005; Murchison 1983; Wang and Reese 1993; Wasserman 2001; Burdette et al. 2004)	Axial and lateral force effects have been considered uncoupled in SPI spring-dashpots representation in different types of foundation soil.
(AASHTO 1996; AISC 1996; Ingram et al. 2003)	The field studies have shown that the prescribed equations in AASHTO and AISC predicted overly conservative values for the ultimate lateral bearing capacity of the piles.
(Novak and Sheta 1980; Wang et al. 1998; Gazetas and Dobry 1984)	Series radiation damping method is more realistic as compared to the parallel radiation damping method.
(Arsoy et al. 2002; Burke Jr 2009)	SIA should be used where rigid soil and long-span bridges are necessary
(Springman et al. 1996; Frangi et al. 2011; Phares et al. 2013; Kerokoski and Laaksonen 2005)	Abutment-backfill soil-approach slab interaction from field studies to mitigate the backfill pressure.

provide uncompromising results to approximate the behavior of an actual integral bridge structure with SSI.

- Backfill and foundation soil density have a very different response to the structural behavior of IAB. The forces and moments in the deck increase and peak moments in the piles decrease when the compaction is varied from loose to dense backfill soil. But for integral abutments on H-piles, the moment in piles increases once the foundation soil changes from loose to dense.
- Approach slab-pavement joint and wingwall parallel to traffic can be used to reduce passive pressures, providing sufficient movement of the integral abutment. Best results can be obtained by monitoring past completed projects before designing them.
- Latest codes like AASHTO (AASHTO LRFD 2012) prescribe the adoption of "Broms" Method (Broms 1964) for the design of piles under lateral loading. Special design criteria for piles below abutments of IABs have not been prescribed anywhere and detailed investigations are mandated with reference to the different geological conditions. More studies should be carried out on the thermal and seismic load combination of IABs (Tsinidis et al. 2019). Considerable researches should be carried out on IABs with short piles, due to the lack of available past studies.
- Till date, the maximum allowable length of IABs is limited to 60–300 m in different geographical strata. Researches should be carried out in the direction to increase the length of IABs with proper design guidelines and experimental procedures for earthquake resistant effects.
- To attain more insight into the SSSI effects on IABs, more numerical and experimental studies need to be carried out. The scientific community should look into ways to alleviate the SSI effects on IABs, rather than to design the piles to sustain large displacements or forces.

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