Thermal Effects on the Structural Performance of Integral Abutment Bridges in Straight and Skew Alignments

Navid Nikravan¹ and Khaled Sennah²

ABSTRACT

This study establishes a practical limit for the maximum length of skewed and non-skewed integral abutment bridges based on displacement-ductility limit state of piles. First, detailed three dimensional (3D) finite element (FE) models were built considering the soil–bridge interaction effects. Then, field measurements from a newly constructed three-span concrete integral bridge with 45° skew angle were used to validate the 3D FE model. Using validated models, a parametric study was carried out to study the effects of skew angle, number of design lane, pile size, bridge length, abutment height and abutment-pile connection on the pile deformation when subjected to seasonal temperature variations. Based on data generated from the parametric study, it was identified that skew angle, number of design lane, piles size, bridge length, abutment height and abutment-pile connection were critical components in pile deformation. Finally, critical parameters were used to develop sets of imperial equations for bridge total length and skew angle limits of integral abutment bridges.

¹Post-Doctoral Fellowship and Former Ph.D. Student, Civil Engineering Department, Ryerson University, 350 Victoria, Toronto, ON, Canada, M5B 2K3.
²Professor and Chair, Civil Engineering Department, Ryerson University, 350 Victoria, Toronto, ON, Canada, M5B 2K3.
INTRODUCTION

Integral abutment bridges are jointless bridges whereby the deck is continuous and monolithic with abutment walls. The use of integral abutment bridges has increased in North America in the past few decades due to their various advantages in economy and safety. Their principal advantages are derived from the absence of expansion joints and sliding bearings in the deck, making them the most cost-effective system in terms of construction, maintenance and longevity. The main purpose of constructing integral bridges is to prevent the corrosion of structure due to water seepage through joints. The simple and rapid construction provides smooth, uninterrupted deck that is aesthetically pleasing and safer for riding. The single structural unit increases the degree of redundancy enabling higher resistance to extreme events. Whereas many integral abutment bridges were built recently in North America, a rational guideline to determine the maximum length limits for non-skewed and skewed integral bridges due to seasonal temperature variations does not exist at present in current AASHTO-LRFD Bridge Design Specifications (AASHTO 2010) and Canadian Highway Bridge Code (CSA 2006). This may be attributed to the lack of research regarding seasonal temperature variation effect on integral bridges. Very few researchers dealt with two-dimensional (2D) and three-dimensional (3D) finite element analysis of integral abutment bridges subjected to thermal loading (among them: Arenas 2012; Baptiste 2011; Christou 2005; Civjan et al. 2007; Fennema, 2005; Huang et al. 2008). Very few research works was conducted to study effect of skew angle on limiting span of integral abutment bridges under temperature variation (Albhaisi et al. 2012; Olson et al; 2013). However, equations for limiting spans of skewed integral abutment bridges with different configurations are as yet unavailable.

Few states and provinces in North America limit span length and skew angle of integral abutment bridge to limit pile lateral displacement due to seasonal temperature variations. As presented in Table 1, policies for maximum integral bridge length and skew angle vary from state to state and maximum integral bridge length is usually limited by empirical data collected from in-service integral bridges. As an example, Tennessee specifies the longest bridge total length limit of 244 m for concrete bridges with no skew angle limitation, while New York specifies the total longest length limit of 200 m for steel bridges with skew angle limitation of 45°. Tennessee also holds the United States record for the longest integral bridge constructed, which has a length of 358 m. However, Ontario allows integral bridges for a total length less than 150 m with skew angle limitation of 20°, irrespective of the bridge material type. The large difference between those limitations clearly indicates that a more rational approach is required to determine the maximum length limit for straight and skewed integral bridges to limit pile displacement due to seasonal temperature variations.

<table>
<thead>
<tr>
<th>State or Province</th>
<th>Maximum length (m)</th>
<th>Skew angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Steel bridges</td>
<td>Concrete bridges</td>
</tr>
<tr>
<td>Ontario (Hussain 1996)</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>Colorado (Diceli 2004)</td>
<td>195</td>
<td>240</td>
</tr>
<tr>
<td>Tennessee ((Diceli 2004)</td>
<td>152</td>
<td>244</td>
</tr>
<tr>
<td>New York (Yannotti 2005)</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>Iowa (Lan 2012)</td>
<td>122</td>
<td>175</td>
</tr>
<tr>
<td>Missouri (Lan 2012)</td>
<td>130</td>
<td>183</td>
</tr>
<tr>
<td>South Dakota (Lan 2012)</td>
<td>107</td>
<td>214</td>
</tr>
<tr>
<td>Vermont (Lan 2012)</td>
<td>119</td>
<td>210</td>
</tr>
</tbody>
</table>

194
In this study, design guidelines are specified to determine more reliable maximum length and skew angle limits for integral bridges based on the horizontal displacement capacity of the steel H-piles. Firstly, a method of estimating pile horizontal displacement capacity for integral bridges was determined. Then, 3D finite-element modeling was developed and validated for different bridge prototypes (1) composite concrete deck slab-over steel girder superstructure and (2) concrete deck slab-over concrete girder superstructure. Using the developed finite-element modeling, a practical-design-oriented parametric study was conducted to determine the effects of selected parameters on pile horizontal displacement. Based on the data generated from the parametric study, imperial equations were derived to determine the realistic length and skew angle limitations of integral bridges to meet the specified limits for pile maximum horizontal displacement under seasonal temperature variation.

**DISPLACEMENT CAPACITY FOR INTEGRAL ABUTMENT BRIDGES**

**DISPLACEMENT CAPACITY FOR NON-SKEWED INTEGRAL ABUTMENT BRIDGES**

Similar to conventional bridges with expansion joints, an integral bridge experiences length changes due to temperature variations. However, conventional and integral bridges accommodate this length change differently. Conventional bridges have a thermally active bridge superstructure and thermally inactive substructure, while integral bridges connect this thermally active superstructure to the substructure. Due to thermal variations throughout the integral bridge service life, the abutment-backfill system and the piles supporting the abutments are subjected to horizontal displacements. This leads to a reduction in the bridge service life due to reduction of load-carrying capacity of piles affected by lateral displacement (Yee 1973). The maximum allowable horizontal displacement of pile without any detrimental effects is referred as the steel H-pile displacement capacity in this study.

The design criteria used by the majority of Departments of Transportation in United States for pile horizontal displacement limits the horizontal movement of a pile with fixed abutment-pile connection at the ground surface to 38 mm in each direction. Further, the value of 38 mm is suggested by Deatherage et al. (2005) as a reasonable allowable horizontal movement of piles with fixed abutment-pile connection based on lateral load tests on H-Piles. The hinged abutment-pile connection compared to fixed abutment-pile connection dramatically increases the displacement capacity of the pile by eliminating damage of the concrete surrounding the pile head (Dicleli 2004; Alhaisi et al. 2012). A value of 46 mm is recommended for pile displacement capacity with hinged abutment-pile connection. Figure 1 shows a schematic diagram of the pile displacement and rotation with increase or decrease of seasonal temperature.
Figure 1. H-pile displacement capacity of integral abutment bridge-soil interface during temperature change.

DISPLACEMENT CAPACITY FOR SKEWED INTEGRAL ABUTMENT BRIDGES

Besides longitudinal displacement under temperature rise and fall, in skewed integral bridge, transverse displacement can occur due to horizontal rotation of the bridge superstructure. Figure 2 shows a plan of skewed bridge superstructure subjected to decrease in temperature resulting in active soil pressure on abutment. An active soil pressure exerts a normal force, $P_a$, on the back of each abutment due to bridge contraction. This force induces a counter-clockwise moment, $M_a$, about Point C which is considered herein as fixed point from which the bridge contracts or expands under temperature variations. This moment is expressed as:

$$M_a = P_a L \sin \theta$$  

(1)

Where $\theta =$ skew angle of the integral abutment bridge; $P_a =$ total active backfill soil force acting against the abutment; and $L =$ total bridge length.

Also, Figure 2 displays that the static friction force between the abutment and the backfill forms a force couple, $M_f$, about point C to resist the horizontal rotation of the bridge superstructure, which is expressed as:

$$M_f = F_{af} L \cos \theta = P_a \tan \delta L \cos \theta$$

(2)

Where $\delta =$ surface-friction angle of soil and abutment with the value of 20° to 26° based on NCHRP Report No. 343 (Barker, 1991).

For the plan view of the skewed bridge shown in Figure 2, the sum of these moments about the point-of-fixity (Point C) of the bridge can be expressed as follows:

$$\Sigma M_c = M_a - M_f = P_a L \sin \theta - P_a \tan \delta L \cos \theta$$

(3)

By inspection, bridge superstructure rotates in-plane when the moment $M_a$ that is induced by the active-soil pressure is larger than the moment $M_f$ that is provided by the soil-frictional force. By setting Equation (3) equal to zero, it can be concluded that rotation of skewed integral
bridge is occurred when the skew angle, $\theta$, exceeds surface-friction angle, $\delta$. Using the above mentioned range for the angle $\theta$, transverse displacements of an integral abutment that lead to bridge rotation need to be considered when the skew angle for an integral abutment bridge is equal to or greater than $20^\circ$.

The criterion to limit the maximum length for skewed integral bridge is the ductility capacity of the abutment piles. Accordingly, the values of pile longitudinal and transverse displacement are considered acceptable when a pile ductility-limit state is satisfied. The displacement-ductility limit state for biaxial bending of an integral abutment pile that was presented by Greimann et al. (1987) is re-written here as Equation (4) based on the bridge superstructure in-plane deformation shown in Figure 3.

$$\left(\frac{\Delta_d}{\Delta_{cx}}\right) + \left(\frac{\Delta_t}{\Delta_{cy}}\right) \leq 1 \quad (4)$$

With,

$$\Delta_d = (dl)\sin\theta - (dt)\cos\theta \quad (5)$$

$$\Delta_t = (dl)\cos\theta + (dt)\sin\theta \quad (6)$$

Where $dl =$ distance of Point A to Point A’ along the longitudinal direction (see Figure 3), $dt =$ distance of Point A to Point A’ along the transverse direction; $\Delta_d =$ total displacement components in the x-axis for a pile, $\Delta_t =$ total displacement components in the y-axis directions for a pile; and $\theta =$ skew angle of the integral abutment bridge. $\Delta_{cx}$ and $\Delta_{cy}$ are considered herein as the displacement capacities of the piles and taken as 38 and 46 mm for abutment-pile fixed and hinged connections, respectively.

THREE-DIMENSIONAL (3D) FINITE ELEMENT MODELING

Three dimensional (3D) structural models of integral abutment bridges, as illustrated in Figure 4, were developed and analyzed using the finite-element–based software (SAP2000
2010) to study the effect of temperature change on pile displacement. In superstructure modeling, shell elements were used to model deck slab, diaphragms, concrete and steel I-girders. Cross-type steel bracing was used in case of steel girder bridge superstructure and modeled using beam elements. Bearing at the pier locations were also modeled to provide only vertical support and all displacements in horizontal directions. The connection between the deck slab and girder was assumed to be fully composite. Rigid links were placed between the centroids of girder top flange shell elements and the centroids of the deck slab shell elements above them in order to satisfy the compatibility of the composite behavior. In the substructure modeling, abutments and wingwalls were modeled with shell elements. The piles were modeled with beam elements. The backfill-abutment and soil-pile interaction models were based on the sub-grade reaction approach, developed by Reese et al. (1974). This approach was based on the Winkler foundation, where the foundation was assumed to be elastic and the soil was modeled as continuous springs. In the developed finite-element model, soil springs had nonlinear resistance displacement relationships represented by two types of interaction curves: backfill-abutment interaction curves and soil-pile interaction curves.

BACKFILL-ABUTMENT INTERACTION MODELING

Since the abutment backfills were considered as a critical part in the entire structural system of integral bridges, soil structure interaction became crucial to ensure that results are well–representative of actual structural behavior. When an integral bridge contracts due to a decrease in temperature, active backfill pressure will immediately develop behind the abutment at a very small displacement. The intensity of this active backfill pressure can be directly calculated using Rankine’s theory. Thus, only active earth pressure needs to be considered under negative thermal variation. However, when the bridge elongates due to an increase in temperature, the backfill pressure coefficient may change between the at-rest backfill pressure coefficient, \( K_{oa} \), and the passive backfill pressure coefficient, \( K_{op} \), depending on the abutment displacement. Hussain and Bagnariol (1996) obtained the relationship between backfill pressure coefficient, \( K \), as a function of abutment movement to the abutment height, as shown in Figure 5, from experimental data and finite-element analyses. This relationship was implemented in the structural model to simulate the backfill-abutment interaction effects under positive thermal variation. In SAP2000 software, multi-linear springs were attached at each node along the abutment to model the force–deformation behavior of the backfill. The stiffness values of the springs were calculated based on Hussain and Bagnariol’s backfill pressure coefficient–displacement relationship. Different spring forces versus abutment displacements for different abutment heights adopted in this study. Spacing of multi-linear soil springs perpendicular to abutment walls was taken as 1 m. In addition to the normal pressure acting against the surface of the abutments, the friction between the abutment and backfill becomes very important in skew alignments. To simulate the friction force between the abutment wall and adjacent soil, a series of multi-linear springs were placed in the direction parallel to the abutment. The stiffness of those springs was obtained by multiplying the stiffness of the normal soil springs with the friction coefficient between the abutment concrete and the backfill. The spacing of the multi-linear springs parallel to the abutment wall was taken close or equal to 0.8 m. Granular uncompacted material with internal friction angle (\( \phi \)) of 30° and unit weight (\( \gamma \)) of 20 kN/m³, which is typically used in integral bridge construction, was assumed for the backfill.
SOIL-PILE INTERACTION MODELING

Two perpendicular horizontal multi-linear springs (i.e. in longitudinal and lateral direction of traffic) were used to model the soil-pile interaction. The soil–pile interaction for a particular point along the pile was defined by a nonlinear load (P)-deformation (Y) curve or P–Y curve, where P is the lateral soil resistance per unit length of pile and Y is the lateral deflection. The computation of the lateral force–displacement response of a pile involved the construction of a full set of P–Y curves along the pile to model the force–deformation response of the soil. It should be noted that two types of soil were considered in this study namely: sand and clay. The method of modeling clay and sand are presented respectively.

For piles driven in clay, a typical P-Y curve is shown with a solid line in Figure 6 (Reese and Van Impe 2001). The P-Y relationship is nonlinear and can be expressed as follow:

\[ \frac{p}{p_u} = 0.5 \left( \frac{y}{y_{50}} \right)^{0.5} \]  

(7)

Where \( p_u \) = ultimate lateral soil resistance per unit length of pile; and \( y_{50} \) = the 50% of the deflection of soil at ultimate resistance, expressed as:

\[ y_{50} = 2.5 \varepsilon_{50} d_p \]  

(8)

Where \( \varepsilon_{50} \) = soil strain at 50% of ultimate soil resistance; and \( d_p \) = width of pile.

As shown in Figure 6, the value of \( \frac{p}{p_u} \) is assumed constant for values of \( y / y_{50} \) equal or greater than 8. Two types of soil behavior are generally considered in estimating \( p_u \) for laterally loaded piles in clay. The first type of behavior occurs near the surface, where the pile may push up a soil wedge by lateral movement, resulting in a so-called wedge action. The second type of behavior occurs at some depth below the ground surface, where the soil attempts to flow around the pile. In case of integral bridges, the backfill and the embankment soil exert surcharge pressures on the foundation soil and may prevent the wedge action. Accordingly, the ultimate soil resistance per unit length of pile, \( p_u \), is expressed as follow:

\[ p_u = 9c_u d_p \]  

(9)

Where \( c_u \) = undrained shear strength of the clay; and \( d_p \) = width of pile.

The non-linear behavior of P–Y curve is simplified using an elasto-plastic curve displayed on Figure 6 with a dashed line. Based on the method proposed by Skempton (1951), the spring stiffness for clay for elastic portion \( K_s \) with consideration of 1 m spring spacing along pile length is obtained as:

\[ K_s = \left( \frac{p_u}{2} \right) / (2.5 \varepsilon_{50} d_p) = 9c_u / 5 \varepsilon_{50} d_p \]  

(10)

For soft, medium and stiff clay, corresponding values of \( c_u \) were considered as 20, 60 and 120 kPa, respectively (Bowles 1996) and corresponding values of \( \varepsilon_{50} \) were considered as 0.02, 0.008 and 0.005, respectively (Evans 1982).
For piles driven in sand, the soil resistance per unit length of pile, \( P_u \), is expressed as:

\[
P_u = k_a d_p (\gamma z + q)(\tan \beta)^8 - 1) + k_o d_p (\gamma z + q)(\tan \beta)^4 \tan \phi
\]  

(11)

Where, \( k_a \) = active earth pressure coefficient; \( k_o \) = at-rest earth pressure coefficient; \( \gamma \) = unit weight of soil; \( \phi \) = angle of internal friction of the soil in degrees; \( z \) = depth below the ground surface; \( q \) = surcharge pressure; and \( \beta \) is expressed as:

\[
\beta = (45 + \phi / 2)
\]  

(12)

For sand, spring stiffness for elastic portion (\( K_s \)) with consideration of 1 m spring spacing along pile length is assumed to increase linearly with depth from the ground surface and expressed as:

\[
K_s = K' z
\]  

(13)

Where \( K' \) = sub-grade constant of the soil.

For loose, medium and dense sand, corresponding values of \( K' \) were taken as 2000, 6000 and 12000 kN/m\(^3\), respectively while corresponding values of \( v' \) were taken as 16, 18 and 20 kN/m\(^3\). Soil friction angles of 30°, 35° and 40° were taken for loose, medium and dense sand respectively (Bowles 1996).

In order to determine a reasonable spacing of soil springs, a sensitivity analysis was conducted (Nikravan, 2013). Based on analysis, when spacing between soil springs along pile length was considered 1 m, the computation errors were less than 1.5%. To keep the accuracy of the solution, it was necessary to limit the spacing of soil springs on the depth below the top soil surface to less than 1 m. More details in this aspect can be found elsewhere (Nikravan 2013).
VALIDATION OF FINITE-ELEMENT MODELING

The field measurements from newly constructed Boone River Bridge (Girton et al. 1991), were used to validate the developed 3D finite element model in this study. The four-span-continuous concrete bridge has a total length of 99 m and width of 12.2 m. Two of the piers are located 24.6 m from each abutment and the third pier is located in the center of the bridge. The superstructure of the bridge consists of 6 prestressed concrete girders and a 190-mm concrete deck slab that forms a skew angle of 45° with the substructure. Each abutment wall is supported on five HP250×85. The piles were driven in a predrilled hole approximately 2.74 m with the strong axis parallel to the longitudinal direction and battered at a slope of 4:1 in the lateral direction only. Ambient temperatures and longitudinal displacements at selected locations in the bridge were recorded by thermocouples and linear variable differential transformers (LVDTs), respectively (Girton et al. 1991). The maximum temperature increase and decrease of +25°C and -47 °C were captured during monitoring program.

Also, finite-element models similar to those as explained earlier were developed for Boone River Bridge to evaluate the longitudinal displacements that were induced by temperature changes. Multi-linear springs were used to model soil surrounding pile and backfill. No springs were provided in the first 2.74 m of pile to represent the existence of predrilled hole just below the abutment. In order to compare results, temperature variations implemented in the finite-element modeling were the same as the measured temperature variation from field testing. Different temperature variations considered for finite-element modeling were +25°C, +12.5°C, -23.5°C and -47°C.

Table 2 presents a good agreement between the results from the field data and the 3D FE model. It is noted that when bridge expansion and contraction occurred the differences between experimental and predicted FEA results for abutment were limited to 10%. For instance, contraction due to temperature decrease of -47°C at top of the abutment obtained from field test was 25 mm, compared to 26.8 mm obtained from finite element analysis (an overestimation of around %7).

<table>
<thead>
<tr>
<th>Case</th>
<th>Temperature Variation</th>
<th>Displacement (mm)</th>
<th>Field Data (Girton 1991)</th>
<th>3D FE Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elongation</td>
<td>ΔT=+25°C</td>
<td>+12.5</td>
<td>+11.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ΔT=+12.5°C</td>
<td>+6.3</td>
<td>+5.7</td>
<td></td>
</tr>
<tr>
<td>Contraction</td>
<td>ΔT=-23.5°C</td>
<td>-12.8</td>
<td>-13.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ΔT=-47°C</td>
<td>-25</td>
<td>-26.8</td>
<td></td>
</tr>
</tbody>
</table>

PARAMETRIC STUDY

A parametric study was conducted to investigate the effects of various geometric and structural properties of bridge components and geotechnical properties of the surrounding soil on the performance of integral bridges subjected to seasonal temperature variations. Table 3 summarizes the range of parameters considered in this study. The parametric study was based on the following assumptions: (i) The average negative and positive temperature ranges for steel integral bridges were considered as -48°C and +40°C and those ranges for concrete bridges were defined as -38°C and +30°C as specified in the Canadian Highway Bridge Design Code (CSA 2006); (ii) Coefficients of thermal expansion of steel and concrete were assumed to
be $1.17 \times 10^{-5}$ and $0.99 \times 10^{-5}$ 1/°C, respectively; and (iii) Modulus of elasticity of the concrete and steel were assumed to be 24 and 200 GPa respectively.

Two 2-lane integral bridges with width of 10.25 m and different girder types, namely: steel and concrete, were chosen as the base bridges to study key parameters affecting their structure response. These bridges had continuous span of 20-m length supported over piers to form the entire bridge length. The first bridge, referred as the steel integral bridge, was composed of four steel I-girders (W760×173) spaced at 2.75 m. The second bridge, referred as the concrete integral bridge, was consisted of four prestressed concrete girders (CPCI 900) spaced at 2.75 m. The superstructure cross-section of above-mentioned bridges was composed of a 250-mm thick concrete slab. In case of concrete bridges, full height diaphragms with 160-mm thickness were considered at bridge mid-span and pier locations. While in case of steel bridges, cross-type bracings (L150×150×25 mm) were spaced at the middle of each span and at pier locations. The abutment walls for bridges were taken 10.25 m long, 1 m thick and 3 m height. Five HP 310x110 steel piles at spacing of 2 m were used to support bridge substructure. Wingwalls of 3 m long; 0.5 m thick and non prismatic depth of 3 m were considered at abutment locations.

### TABLE 3 VARIABLES CONSIDERED IN THE PARAMETRIC STUDY

<table>
<thead>
<tr>
<th>Variables</th>
<th>Range of variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skew angle and number of design lanes</td>
<td>Skew angles: 15, 20, 30, 45, 60°</td>
</tr>
<tr>
<td></td>
<td>Number of lanes: 1, 2, 3, 4, 5, 6, 7, 8</td>
</tr>
<tr>
<td>Pile size and bridge length</td>
<td>Sizes: HP 200x53, HP 250x85, HP 310x110</td>
</tr>
<tr>
<td></td>
<td>Bridge lengths: 20, 100, 180 m</td>
</tr>
<tr>
<td>Abutment height and abutment-pile connection</td>
<td>Abutment height: 3, 5 m</td>
</tr>
<tr>
<td></td>
<td>Connections: Fixed, Hinged</td>
</tr>
</tbody>
</table>

### EFFECT OF SKEW ANGLE AND NUMBER OF DESIGN LANES

For the purpose of realizing the influence of skew angle, behavior of steel integral bridges with different skew angles ranging from 0 to 60° during temperature fall were studied. It is observed in Figure 7 that the skew angle of bridge had a remarkable effect on the temperature-induced transverse pile displacement. Maximum pile transverse displacement generally increased with increasing skew under both expansion and contraction. During temperature fall, for a 60° skewed integral bridge compared to 30° skewed integral bridge, pile transverse displacement increased by 71% (from 13.03 to 22.3 mm). The fact worthy of attention is that in the case of integral abutment bridges with skew angles less than surface-friction angle between the abutment and the backfill ($\theta \leq 20°$), transverse pile displacements were equal to zero.

For evaluation the influence of number of design lanes, steel integral bridges with 100 m total length and different widths (7.00, 10.25, 13.5, 16.75, 20.00, 23.25, 26.5 and 29.75 m representing bridges with 1 to 8 design lanes) were analyzed. By inspection of the finite element analysis results shown in Figure 8, it was found that a change in the number of design lanes significantly affected the transverse displacements of an integral abutment when the bridge-skew angle, $\theta$, exceeds about 20°. For angles $\theta$ greater than about 20°, an increase in the number of design lanes, n, caused an increase in the transverse displacement. As an example, for the one-design-lane steel bridge, the transverse pile displacement at a skew angle of 60° was
approximately 20.15 mm while for the eight-design-lanes integral bridge, the transverse displacement at a skew angle of 60° was approximately 27.08 mm. It is noteworthy to mention that the integral bridges with the same length irrespective of different number of design lanes had the same values of longitudinal displacements.

**EFFECT OF PILES SIZE AND BRIDGE LENGTH**

In order to determine the effect of pile size, behavior of non-skewed steel integral bridges with 100-m total length and different H piles (HP 200×53, HP 250×85 and HP 310×110) were compared. Figure 9 illustrates that smaller pile-head displacement occurred for stiffer piles during both temperature rise and fall. For instance, pile-head displacement remarkably decreased by 18% (from 22.4 to 26.4 mm) when the pile size changed from HP 310×110 to HP 200×53. The reason of smaller pile displacement was attributed to that larger pile flexural stiffness brought larger constraints to the expansion and contraction of the superstructure. It should be pointed out that pile-head displacements were larger under contraction than under expansion. For HP 250×85, the pile displacement was equal to 23.96 mm during temperature fall and was equal to -12.65 mm during temperature rise.

Three non-skewed steel integral bridges with different total lengths (20, 100 and 180 m) were analyzed to evaluate the effect of increasing total bridge length on the integral bridge behavior under negative thermal loading. Figure 10 provides sample results for a 310×110 H-Pile oriented for strong axis bending in medium clay. It illustrates that pile horizontal displacement of 7.3 mm for 20 m integral abutment bridge length increased to 23.1 and 37.8 mm with increase in total bridge length to 100 and 180 m, respectively. It can be concluded that pile displacements increase considerably as the bridge becomes longer.
EFFECT OF ABUTMENT HEIGHT AND ABUTMENT-PILE

Non-skewed steel integral bridges with 160-m total length and different abutment heights (3 and 5 m) were chosen to investigate the effect of the abutment height. It is observed from Figure 11 that the horizontal pile displacement decreased notably with increasing the abutment height. As an example, pile displacement was reduced from 37.8 mm to 33.6 mm by changing abutment height from 3 to 5 m (a decrease of 13%).

In order to examine the effect of abutment-pile connection, non-skewed steel integral bridges with different lengths (20, 100 and 180 m) with hinged- and fixed-head conditions were selected. Figure 12 illustrates that the maximum bending moment occurred below the pile head for pile hinged-head conditions, as expected, while these values reached their maximum at the pile head for fixed-head conditions. It is also observed that maximum bending moment for hinged connections were noticeably less than that for fixed connection. For instance, in a steel integral bridge with total length of 100 m, maximum bending moment with a pile fixed-head condition was 113.21 kN. m, while this value with pile hinged-head condition was 78.77 kN. m (reduction of around 44%). Certainly, to have a longer integral bridge, hinged connection is recommended since the pile bending moment is smaller than those for the pile fixed-head condition. In addition, the moment through the hinged connection would not be transferred to the superstructure as the case for pile fixed-head condition.
RECOMMENDED MAXIMUM LENGTH LIMITS FOR INTEGRAL BRIDGES

In the parametric study, the critical pile displacements were first computed for bridges based on different combinations of critical variables including abutment height, piles size and abutment-pile connection. Then, those critical pile displacements were used to determine length and skew angle limits for integral bridges. Maximum acceptable lengths for integral bridges were the longest lengths in which the critical pile displacements obtained in the bridges were less than the pile-displacement capacity demands explained in previous sections.
Figure 13 depicts the bridge limiting length as a function of the number of design lanes along with the change in skew angle. It can be observed that for skew angles larger than about 20°, an increase in the number of design lanes caused a decrease of nearly 3% in allowable length while for skew angles smaller than 20° an increase in the number of design lanes had no effect in allowable length.

Furthermore, the bridge limiting length as a function of the abutment height is shown in Figure 14 with solid lines. The nonlinear relationship between bridge limiting length and abutment height was simplified using a quadratic curve displayed on the same figure with dashed lines. The use of quadratic curve made us to be in a safe side for design in a way that the values obtained from quadratic curve for allowable total bridge lengths were smaller than nonlinear ones. Using statistical package for curve fit, new sets of imperial equations for integral bridges were deduced to determine length limits as a function of the skew angle, abutment height, pile size, abutment-pile connection type and number of design lanes. These equations are listed in Tables 4 for steel and concrete bridges, respectively.

![Figure 14. Allowable lengths for steel bridges with different abutment heights and fixed abutment-pile connection.](image)

Figure 15 shows the correlation of imperial expressions with the corresponding values as obtained from the finite element analysis for the maximum length limits. It can be observed that developed equations correlate very well with the FEA results. Maximum length limits obtained from equations and FEA differed by less than 5%. As an example, maximum length limit for steel integral bridge with 2 design lanes, 3 m abutment heights, 30° skew angle, 310×110 H-piles and fixed abutment-pile connection were computed as 98.75 m and 100 m based on proposed equation and FEA results (around only 1% difference).
## TABLE 4 ALLOWABLE LENGTH FOR SKewed INTEGRAL ABUTMENT BRIDGES

<table>
<thead>
<tr>
<th>Bridge type</th>
<th>Abutment pile connection</th>
<th>Skew angle, θ (degrees)</th>
<th>Maximum length, L (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fixed</td>
<td>0≤θ≤20</td>
<td>L=[-0.04θ²+0.01θ] + (-0.4H²+11H+199.3) [1-0.03(n-2)]</td>
</tr>
<tr>
<td></td>
<td>Hinged</td>
<td>20&lt;θ≤60</td>
<td>L=[-0.03θ²-1.03θ] + (-0.4H²+11H+289.3) [1-0.03(n-2)]</td>
</tr>
<tr>
<td>Steel bridges</td>
<td>Fixed</td>
<td>0≤θ≤20</td>
<td>L=-0.4H² +11H+145</td>
</tr>
<tr>
<td></td>
<td>Fixed</td>
<td>20&lt;θ≤60</td>
<td>L=-0.4H² +11H+15+111H+179.5 [1-0.03(n-2)]</td>
</tr>
<tr>
<td></td>
<td>Hinged</td>
<td>0≤θ≤20</td>
<td>L=-0.4H² +11H+135</td>
</tr>
<tr>
<td></td>
<td>Hinged</td>
<td>20&lt;θ≤60</td>
<td>L=[(-0.01θ²-1.68θ) + (-0.4H²+11H+123)] [1-0.03(n-2)]</td>
</tr>
<tr>
<td>Concrete bridges</td>
<td>Fixed</td>
<td>0≤θ≤20</td>
<td>L=-0.4H² +11H+165</td>
</tr>
<tr>
<td></td>
<td>Fixed</td>
<td>20&lt;θ≤60</td>
<td>L=[(-0.04θ²+0.01θ) + (-0.4H²+11H+159.5)] [1-0.03(n-2)]</td>
</tr>
<tr>
<td></td>
<td>Hinged</td>
<td>0≤θ≤20</td>
<td>L=-0.4H² +11H+175</td>
</tr>
<tr>
<td></td>
<td>Hinged</td>
<td>20&lt;θ≤60</td>
<td>L=[(-0.03θ²-1.03θ) + (-0.4H²+11H+239.3)] [1-0.03(n-2)]</td>
</tr>
</tbody>
</table>

Note: n = number of design lanes, H = abutment height in meter (1 m ≤ H ≤ 5 m).

![Figure 15](image-url)  
Figure 15. Correlation between FEA results and those obtained from the developed equations for bridge total length limits.
CONCLUSIONS

Based on the results obtained from the parametric study conducted on selected integral abutment bridge configurations subjected to seasonal temperature variations, the following conclusions are drawn:

• Abutments height has a significant influence on pile displacement. Increase in abutment height is recommended to increase the length limits of integral bridges to sustain thermal displacements.
• Bridges with skew angle greater than 20° experience pile transverse movement. Skewed bridges with skew angles less than 20° exhibited insignificant pile transverse movement under seasonal temperature variations.
• Increase in the number of design lanes or bridge width significantly increases pile transverse displacement and hence decreases total bridge length limit of skewed bridges with skew angle greater than 20°.
• Larger H-pile sizes in integral abutment bridges lead to noticeably smaller pile-head displacement and smaller pile-head moment.
• A concrete bridge has longer allowable total length than that for a steel bridge as the former is less sensitive to seasonal temperature variations.
• Sets of imperial equations to determine bridge limiting length to ignore effects of temperature variations on pile horizontal displacement are developed for concrete and steel bridges in the form of steel H-pile size, abutment height, abutment-pile connection type, skew angle and number of design lanes. This set of imperial equations has the following scope and limitations of use: (i) number of design lanes ranges from 1 to 8; (ii) the abutment-pile joint can be hinged or fixed; (iii) the developed equations are applicable to slab-on-girder bridges made of concrete or steel materials; (iv) the developed equations are applicable to three H-pile sizes, namely: HP 200 ×53, HP 250 ×85, and HP 310 ×110; (v) the bridge skew angle does not exceed 60°; and (vi) abutment height ranges from 1 to 5 m.

ACKNOWLEDGMENTS

The authors wish to acknowledge the funding received from NSERC Discovery Grant and the Highway Infrastructure Innovation Funding Program of Ontario Ministry of Transportation (MTO–HIIFP). Opinions expressed in this paper are those of the authors and do not necessarily reflect the views and policies of the Ministry.

REFERENCES

C


Yee, W.S. (1973). Lateral Resistance and Deflection of Vertical Piles - Phase I. Bridge Department, Division of Highways, California Department of Transportation, Sacramento, CA, USA.