Two-Dimensional Performance of Integral Abutment Bridges using Finite Element Analysis

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ABSTRACT
Integral Abutment Bridges are complex structures designed to encompass a continuously rigid assembly deprived of expansion and bearing joints. Under cyclic thermal loading, the structure operates as one unit through expanding and contracting mechanisms giving rise to passive and active earth pressures, respectively. To fully understand this compound response, a Two – Dimensional Finite Element model of the SR – 18 over Mississinewa Bridge was developed using PLAXIS. A hardening soil model was utilized to simulate the hysteretic nonlinear elasticity and stress dependent stiffness behavior of soils. This model aims to replicate the behavior of the structure under current conditions. Simulations were validated with field measured deformations thus providing an operational model capable of simulating the long-term performance of the bridge. This paper provides the preliminary results obtained following the FE analysis where they will be supported by further details that are to be discussed over the course of the conference.

RÉSUMÉ
Les ponts intégrés d'abutment sont des structures complexes conçues pour englober un assemblage continuellement rigide privé d'expansion et de joints de roulement.Sous la charge thermique cyclique, la structure fonctionne comme une seule unité grâce à l'expansion et la passation de mécanismes de contraction donnant lieu à des pressions passives et actives de la terre, respectivement.Pour bien comprendre cette réponse composée, un modèle d'élément fini bidimensionnel du SR – 18 sur le pont Mississinewa a été développé à l'aide de PLAXIS.Un modèle de sol durcissant a été utilisé pour simuler l'élasticité non lignetaire hystérétique et le comportement de rigidité dépendant du stress des sols.Ce modèle vise à reproduire le comportement de la structure dans les conditions actuelles.Les simulations ont été validées avec des déformations mesurées sur le terrain, fournissant ainsi un modèle opérationnel capable de simuler les performances à long terme du pont.Ce document fournit les résultats préliminaires obtenus à la suite de l'analyse fe où ils seront appuyés par d'autres détails qui doivent être discutés au cours de la conférence.

1 INTRODUCTION

Bridge design has been known to be one of the oldest methodologies in Structural and Geotechnical Engineering. A typical bridge is designed to fulfill serviceability and structural requirements. The serviceability of a bridge requires it to successfully connect two locations at a distance from each other thus providing a passageway in the presence of an obstruction as well as maintaining their degree of deflection (Balasubramanian, 2017). This will provide a safe and stable platform for the public. The structural requirement of a bridge involves the adequate resistance of applied external forces such as: Dead Loads, Live Loads, Snow Loads, Wind Loads, Thermal Loads, and Earthquake Loads. This research concentrates on the behavior of such structures under cyclic thermal loading. Thermal loads are temperature induced forces that occur due to the seasonal changes in the ambient temperatures. During cold temperatures, the structure's material experiences internal shrinkage therefore leading to the contraction of the bridge. On the other hand, during warmer temperatures, the opposite occurs where the structure undergoes expansion. Conventional bridges resist such movements due to the presence of expansion joints. Expansion joints are connecting points located along the length of the bridge as well as at the supports accommodating the contraction and expansion of the bridge. Even though this has been a common practice for many years, studies have shown that this type of bridge design is uneconomic (Chovichien, 2004). With recent technological advancements, Engineers were able to come up with an innovative design also known as an Integral Abutment Bridge (IAB) that eliminates the use of expansion joints. As a result, the structure resists thermal loads by contracting and expanding away and towards the abutment backfill as a whole unit. These movements will consequently induce increased lateral stresses within the abutment backfill (Tan , Reid, Rajeev, Piratheepan, & Sivakugan, 2014).

IABs have been a recent discovery with no full and comprehensive understanding of their behavior under various conditions. To improve the design of IABs, their behaviour under different loading conditions must be obtained and understood. This is done through creating a software model that allows for the simulation of current loading conditions. For this paper, a two-dimensional model was developed in order to simulate the generated lateral stresses behind the bridge abutment given. Output parameters will be compared to readings obtained from instrumented (at the same level) strain gauges that were utilized during a field investigation. This paper presents the initial results and more details will be shared during the conference.

2 INTEGRAL ABUTMENT BRIDGES

An IAB is a type of structure that encompasses a rigid design. To achieve this rigidity, changes have been made to conventional bridge designs. The main difference between a typical bridge and an IAB is the absence of expansion joints (Chovichien, 2004). Expansion joints have been seen to increase maintenance costs of the structure due their exposure to moisture and salt that cause the joint to erode as well as damages caused by snow plows (Chovichien, 2004). Furthermore, the presence of expansion joints causes structural elements to be more susceptible to water damages (Deshnur, Shreedhar, & Spandana, 2016). The presence of expansion joints was observed to have worsen driving experiences and comfort due to the discontinuity of the structure (Deshnur, Shreedhar, & Spandana, 2016).

The other difference between IABs and conventional bridges is the design of the structure's foundation. IABs utilize a single row of piles oriented about the weakest axis as compared to the multiple rows of piles found in conventional bridges (Chovichien, 2004). As previously mentioned, IABs resist thermal loads through expansion and contraction of the entire structure. This bulk movement results in the stress generation within the foundation of the structure (Frosch & Lovell, 2011). Thus, the utilization of a single row of piles allow for an easier construction process as well as reduced form costs. Orienting piles about the weak axis maximizes flexibility as well as lowering stress generation.

Although the construction of an IAB has been practiced for several years, their complex and unique soil – structure interaction is yet to be understood and is a challenge for structural and geotechnical engineers. When the abutment wall contracts away from the soil backfill during active phases, soil has been seen to settle behind the abutment thus developing a soil wedge at the interface between the abutment and the backfill (Khodair, 2009). Due to this soil deposition, the full expansion of the abutment during passive phases is hindered. Over the long – term repeated expansion and contraction of the structure, the soil wedge behind the abutment experiences large enough strains to cause permanent straining of the soil (Khodair, 2009). This concept is known as soil ratcheting. Moreover, with every completed cycle of loading, it can be seen from the recorded pressures that there is an annual increase in the generated earth pressure (Horvath, 2004). The long – term increase in earth pressure behind the abutment wall gives rise to two potential modes of failure (Horvath, 2004). The first potential failure that may occur incorporates the failure of the soil wedge during passive states (Horvath, 2004). IABs are typically designed for at rest (K_o) conditions meaning that any passively induced pressures are resisted by the utilized factor of safety of the structure. However, as the pressure continues to increase more significantly in the upper region, the theoretical limit of passive pressure may be exceeded thus leading to failure of the soil wedge (Horvath, 2004). The second potential failure that may occur also corresponds to the upper region of the abutment (Horvath, 2004). This involves the formation of a heave at the connection point between the approach slab and abutment wall as the abutment wall pushes against the backfill. This formation of a heave induces additional stresses within the approach slab therefore causing it to failure in flexure (Horvath, 2004).

3 FULL-SCALE MONITORING OF AN INTEGRAL ABUTMENT BRIDGE

3.1 Background

The SR – 18 over Mississnewa river bridge is an IAB located in Marion, Indiana, USA. This bridge encompasses a width of 14.63 m (48 ft), a total span length of 114.60 m (367 ft) consisting of two spans having a length of 18.90 m (62 ft), and three spans having a span of 24.69 m (81 ft). The skewness of the bridge was seen to be 8^o. Furthermore, the superstructure consisted of a 0.2032 m (8 in) reinforced concrete deck overlying five pre–stressed concrete bulb tee girders equally spaced at 3.10 m (10.167 ft). Bents 1 and 6 were constructed using reinforced concrete and were seen to be 2.76 m (108.5 in) tall, 14.63 m (48 ft) wide, and 0.99 m (3.25 ft) thick. Each abutment accommodates a single row of 10-0.356 m (14 in) Concrete Filled Steel tubes (CFTs) spaced at 1.5621 m (5.125 ft). The average pile length in Bent 1 was seen to be 6.34 m (20.8 ft), whereas the average pile length found in Bent 6 was seen to be 8.23 m (27 ft).

3.2 Indiana standards

IAB design methodology is strongly dependent on engineering judgement and experience (Frosch & Lovell, 2011). Thus, construction and design processes differ from one state to another (Frosch & Lovell, 2011). IAB construction in the state of Indiana, US is done with accordance to set recommendations and limitations proposed by the Indiana Department of Transportation (INDOT) (Frosch & Lovell, 2011). The INDOT have placed limits on: Highway Alignment Across the Bridge, Maximum skew angle in degrees, Maximum Bridge Length in feet, and Maximum zero point in feet (Frosch & Lovell, 2011). Limitations established by the INDOT depend on utilized structure type. For reinforced concrete slab structures, no restrictions are set for the highway alignment and the maximum skewness however are limited to a maximum bridge length of 500 ft and a maximum zero point of 250 ft. Structural steel structures are restricted to a tangent only alignment across the bridge, a 30º maximum skew angle, a maximum length of 500 ft, and a maximum zero point of 250 ft. Finally, prestressed concrete structures have no restrictions on the highway alignment across the bridge, a maximum skew angle of 30º, a maximum length of 500 ft, and a maximum zero point of 250 ft (Frosch & Lovell, 2011). Pile foundations supporting the superstructure are designed to only resist gravity loads and utilize a single row of H – piles or CFT piles oriented about the weak axis (Frosch & Lovell, 2011). The INDOT provides two typical design abutment cross-section details.

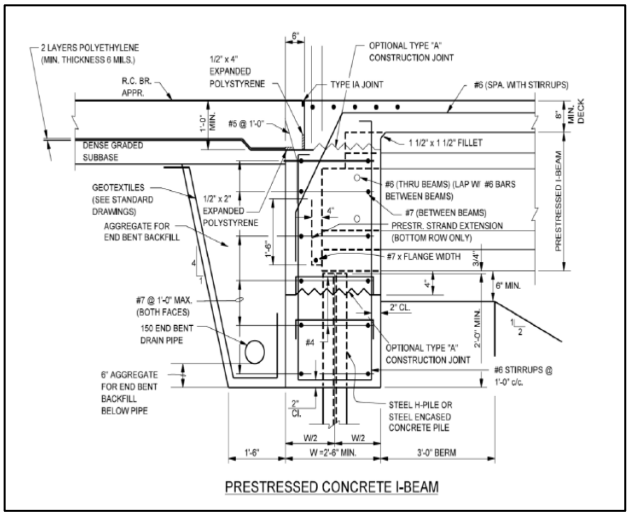

Figure 1-Typical Abutment Detail A *(Frosch & Lovell, 2011)*

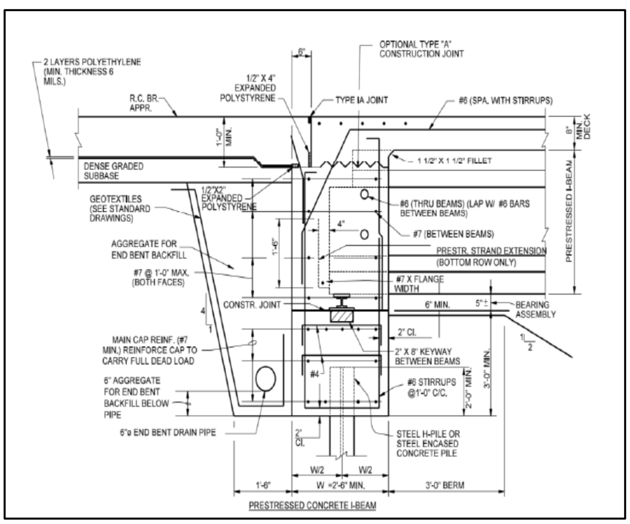

Figure 2-Typical Abutment Detail B *(Frosch & Lovell, 2011)*

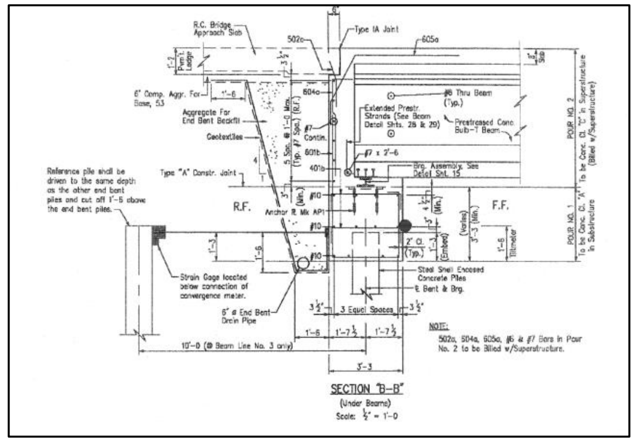

Figure 3-SR-18 Abutment Cross – Section *(Chovichien, 2004)*

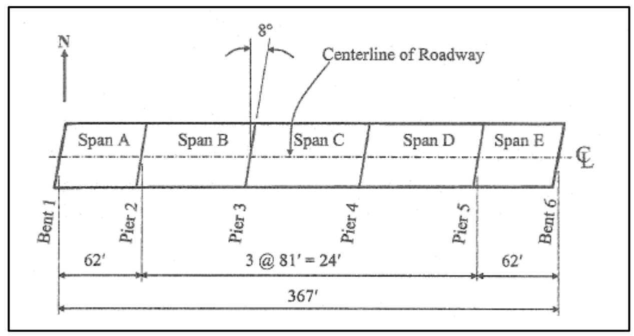

Figure 4-Plan view of SR-18 Bridge *(Chovichien, 2004)*

3.3 Bridge Instrumentation

The SR – 18 bridge was instrumented with four types of data collecting devices in the aim to obtain field readings on key output parameters under the fluctuation of the ambient temperatures. The devices installed included: Earth pressure cells, Convergence meters, Strain gauges, and Tiltmeters. Earth pressure cells, and convergence meters are installed at the abutment wall – backfill interface to monitor the structure deformation under cyclic loading. Earth pressure cells are installed on a bridge to measure the generated pressure behind the abutment wall. As the abutment expands and contracts, induced pressures result in a differential pressure within the cell. This unequal pressure initiates an electrical signal that is perceived at the receptor location correlating to pressure reading (Earth Pressure Cells, 2015). Convergence meters are employed to monitor abutment displacements. Their operating mechanism consists of measuring the relative movement of two distinct locations using either wire extensometers or convergence tapes (Stacey & Wrench, 1985). Finally, a tiltmeter consists of two electrolytic surfaces that are placed normal to each other as well as a temperature sensor. Under deformation, the two orthogonal surfaces generate an electrical signal that measures the degree and direction of tilt (Battaglia, et al., 2019). As shown in Figure 5, a series of earth pressure cells were installed at three locations. The first and second locations can be seen to be along the center of Bent 1 and Bent 6, respectively. The third pressure cell is located on the edge of Bent 6. All Earth pressure cells are placed at an elevation of 0.381 m (1.25 ft) above ground level (Chovichien, 2004). Convergence meters are located along the center of Bent 1 and 6 at an elevation of 0.381 m (1.25ft).

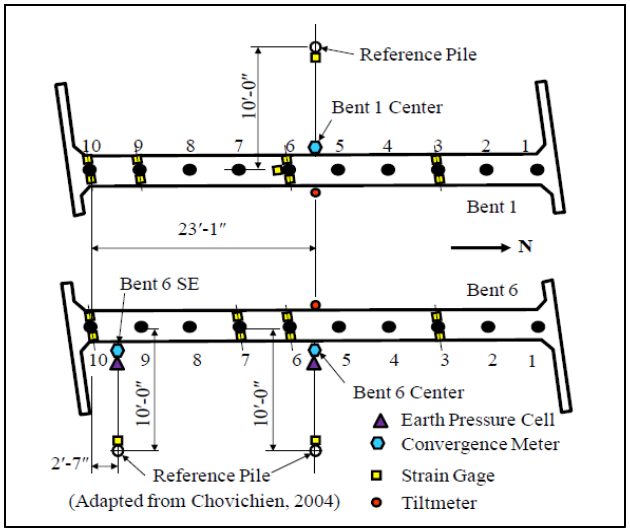

Figure 5-Plan view of bridge Instrumentation *(Chovichien, 2004)*

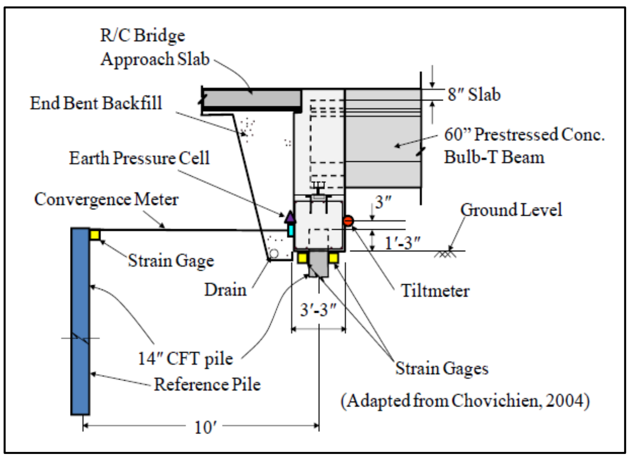

Figure 6-Elevation of instrumentation *(Chovichien, 2004)*

3.4 Summary of Field Results

Following the instrumentation of Convergence meters and Earth pressure cells, the abutment movements as well as generated stresses were monitored for a period of seven years, respectively. The obtained results from this field investigation are summarized in Figure 7 and Figure 8 showing the generated earth pressures and abutment displacements against the corresponding year for all earth pressure cell and convergence meter locations.

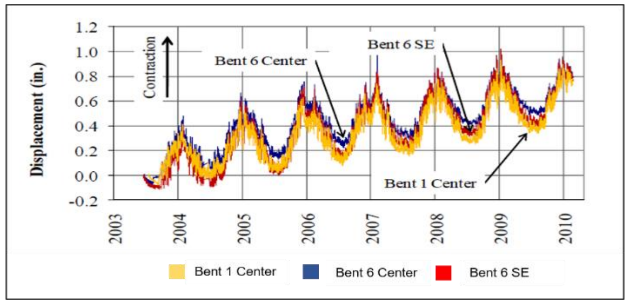

Figure 7-Abutment wall displacement (1 in = 0.0254 m) *(Chovichien, 2004)*

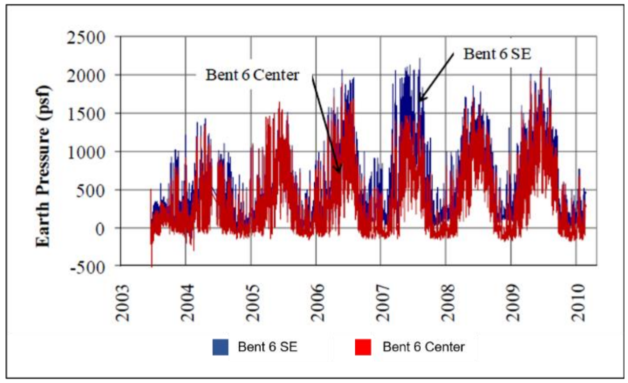

Figure 8-Earth pressure generation in abutment Backfill (1 psf = 0.0479 kPa) *(Chovichien, 2004)*

4 DEVELOPMENT OF FE MODEL

The purpose of the Finite Element Analysis was to develop a functional model that is able to successfully simulate the behaviour of an IAB under cyclic loading obtained from the field. This involved comparing obtained earth pressures from the FE model at the location of the earth pressure cells to those obtained during the field investigation.

4.1 FE Model Description

As part of the analysis, a Two–Dimensional model was developed in order to simulate the behaviour of the structure under cyclic loading. The development of the model consisted of defining soil parameters and stratigraphy, structural geometries and materials, and loading conditions as seen in the field. The Hardening soil model was opted to simulate soil behaviour under loading. Since soil is a nonlinear material, assuming a linear – elastic response under subjected deformation gives rise to inaccuracies that lead to unrealistic results. This would be the case if the Mohr – Coulomb model were utilized. The hardening soil model is a nonlinear model type that was formulated in order to accurately mimic the stiffness-stress dependency of soils, shear and volumetric plastic deformation mechanisms, and dilatancy of soils. This model exhibits accurate variations in stiffness for small shear strain magnitudes through a hysteretic behaviour (El Naggar et al., 2016). As a result, utilizing this model type accurately captures and simulates the soil-structure interaction within the bridge. Figure 9 presents the model plot of the developed system showing the soil profile, pile foundation, bridge abutment, soil backfill, and approach slab.

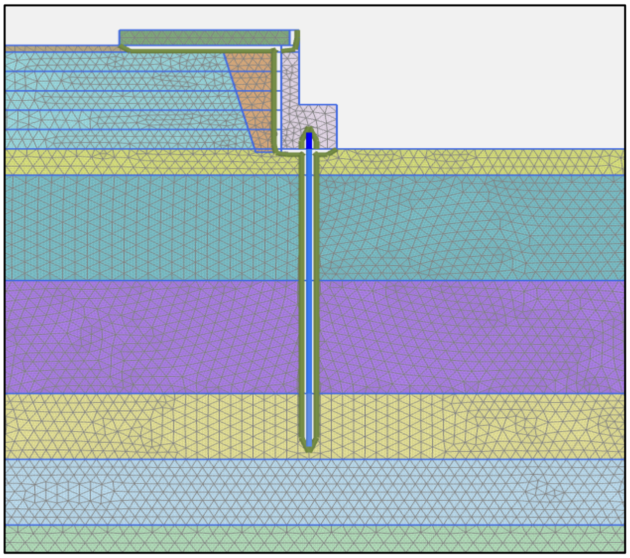

Figure 9-Connectivity plot of model

4.2 Soil and Structural properties

According to the relative density provided in the description of the abutment backfill by (Frosch & Lovell, 2011), necessary parameters were obtained using empirical relationships found in the literature for cohesionless soils. The parameters of the soil layers surrounding the pile were obtained using provided Standard Penetration Numbers as given in the borehole logs. These parameters are summarized in Table 1. The structural components of the model include the abutment, RC deck, and supporting piles. The abutment and RC deck were modelled as volume elements utilizing an elastic and nonporous material type as seen in Table 2. Due to the three-dimensional orientation of piles, a transformed equivalent pile section was required for a two-dimensional model. This included determining an equivalent axial (EA_{eq}) (kN/m) and bending rigidity (EI_{eq}) (kN.m^2/m) by the following Eq.1 and Eq. 2: (El Gendy & El Naggar, 2012):

$$(EA)_{eq} = (n\ (E_pA_p))/((n-1)\ S) \qquad [1]$$

$$(EI)_{eq} = (n\ (E_pI_p))/((n-1)\ S) \qquad [2]$$

where: n is the number of piles, E_p is the pile elastic modulus, A_p is the area of the pile, I_p is the pile moment of inertia, and S is the pile spacing. The CFT piles were modelled as isotropic plate elements behaving elastically where the axial and bending rigidities are presented in Table 3.

Table 1-Soil properties

Material	ϕ^o	c' (kPa)	E_{50} (MPa)	E_{oed} (MPa)	E_{ur} (MPa)	γ (kN/m^3)
Silty Clay	20	20	18	18	54	17.55
Silty Clay	15	20	12	12	36	17.55
Silty Clay	10	20	7	7	21	17.55
Soft Moist Silty Loam	10	15	3	3	9	16.40
Soft Silty Loam	30	0	9.9	9.9	30	18.70
Hard Dry Silty Loam	44	0	20.7	20.7	62	22.30
Backfill	35	0	35	35	105	20.42
Crushed Stone	36	0	90	90	270	20.00

Table 2-Concrete parameters

Material	E (MPa)	γ (kN/m^3)	ν (poissons ratio)
Abutment	22106	24	0.15
Approach Slab	23632	24	0.15

Table 3-Pile parameters

Material	$(EI)_{eq}$ (kN.m^2/m)	$(EA)_{eq}$ (kN/m)	ν (poissons ratio)
Pile	18789	24	0.15

4.3 Staged Construction

In order to accurately model the behaviour of a structure using a software, the definition of a construction process is key. This involves splitting the calculation process into separate phases with each phase encompassing a different loading condition. The first calculation phase is known as a parent phase which is followed by a series of child phases. By default, a newly defined phase will proceed its calculation activity according to the deformations resulting from the previous phase. As a result, it is important to properly outline a reasonable construction sequence to attain a stress history corresponding to that adopted in the site. An inaccurate construction sequence may compromise the reliability of the results. For this analysis, a construction sequence was defined according to the construction records of the bridge:

1. Initial condition of stress generation due to weight of soil and water conditions set. The K_o procedure is selected where vertical and horizontal stresses are formulated by Eq. 3 and Eq. 4, respectively:

$$\sigma_v = h \times \gamma \qquad [3]$$

$$\sigma_h = k \times \sigma_v \qquad [4]$$

where h is the thickness of the soil layer and k is the coefficient of earth pressure

2. Equivalent Pile is driven into the ground to a depth of 6.86 m (22.5 ft) leaving a length of 0.381 m (1.25 ft) to be embedded into the abutment pile cap
3. Abutment concrete is poured and a rigid connection between pile and pile cap is developed
4. First layer of backfill is placed behind abutment
5. Second layer of backfill is placed behind abutment
6. Third layer of backfill is placed behind abutment
7. Fourth Layer of backfill is placed behind abutment
8. Fifth layer of backfill is placed behind abutment
9. 0.152 m (6 in) thick layer of crushed stone is placed over fifth layer of backfill
10. 0.356 m (14 in) thick Approach slab is poured and formed
11. Axial loads from abutment dead load, bridge deck, girder, and live load is applied along with prescribed cyclic lateral predefined displacements at the location of the convergence meters (7.32 m,0.381 m)

Table 4- Average cyclic displacements

Year	Displacement change (mm)	Displacement Direction
Jan-04	7.8	Towards Backfill
Jun-04	7.6	Away from Backfill
Jan-05	9.8	Towards Backfill
Jun-05	8.0	Away from Backfill
Jan-06	9.2	Towards Backfill
Jun-06	5.9	Away from Backfill

4.4 Results and Discussion

As seen in Figure 10, a comparative analysis was carried between results obtained from field measurements and results obtained using PLAXIS 2D. Following this data acquisition, the following observations were seen over a period of two and a half years from January 2004 till June 2006. The earth pressures presented using PLAXIS can be seen to adhere to that shown by the pressure cells. Moreover, the model was successfully able to capture and represent the phenomenon known as soil ratcheting as supported by the model outputs. As presented in Figure 10, during the beginning of the year 2004, the recorded earth pressure by the pressure cell during the active phase was seen to be 43.09 kPa (900 psf) similar to that tabulated using the developed model which came to a value of 38.88 kPa (812 psf). As ambient temperatures increase, the structure experienced a thermal expansion thus increasing the generated pressure to a value of 69.42 kPa (1450 psf) as recorded by the pressure cell and a value of 67.03 kPa (1400 psf) as generated using PLAXIS. The generated pressure then experiences a drop in magnitude to the contraction of the structure where the recorded pressure by the pressure cell was seen to be 28.72 kPa (600 psf) and 23.94 kPa (500 psf) for the years 2005 and 2006, respectively. This was simulated by the model as shown in Figure 10 to have generated pressure values of 20.10 kPa (420 psf) and 16.27 kPa (340 psf) for the years 2005 and 2006, respectively. Similarly, the passive pressures recorded were seen to be 71.82 kPa (1500 psf) and 73.01 kPa (1525 psf) for June 2005 and 2006 respectively. Pressures generated from the model were once again similar to that of the field readings were the obtained values were seen to be 70.38 kPa (1470 psf) and 67.03 kPa (1400 psf) in the June of 2005 and 2006, respectively.

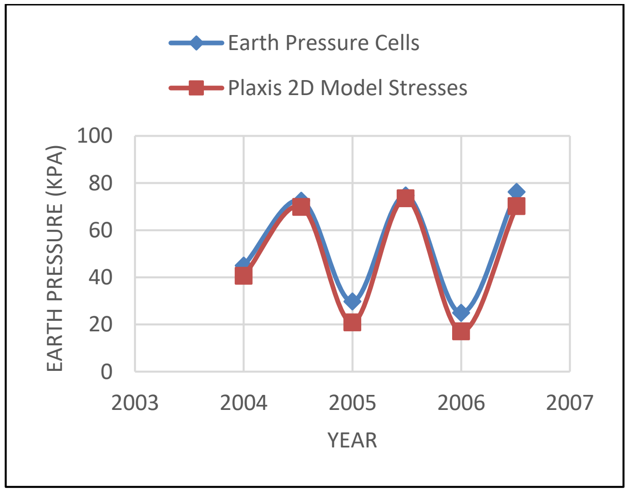

Figure 10-Earth pressure as seen by pressure cells compared to PLAXIS 2D

5 SUMMARY AND CONCLUSION

Over the years, the utilization of IABs has been seen to increase in popularity due their unique design and performance. These structures are able to resist cyclic lateral loading due to the variation in the ambient temperatures as well as resist axial loads due to structural component dead loads and live loads through moment distribution to the supporting foundation. As part of their exceptional design, these structures were able to eliminate the necessity of expansion joints thus reducing construction complexity and maintenance costs as well as reducing the construction time. Nonetheless, the soil – structure interaction of IABs is yet to be fully understood and further research is required. This research aids in the understanding of IABs as it provides a platform that could be used to grasp the long – term behaviour of these structures allowing engineers to dive a deeper in understanding the soil – structure interaction. To perform this research, information regarding the site conditions were acquired to closely simulate field conditions, proper definition and modelling of structural elements was key to accurately mimic the soil-structure interaction, and to apply cyclic loading conditions to that experienced in the field. It can be seen from the results provided that employing a two-dimensional analysis as well as the hardening soil model, this model was successfully able to simulate the nonlinear behaviour of the soil and the SR-18 over Mississinewa river. In the second phase of this design project, pile performance with climate change will be analyzed based on this study. The change in pile behaviour as a result of varying key parameters will also be studied to further understand the behaviour of piles. This shall be accompanied by providing engineers with a comprehensive set of the necessary guidelines.

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