

Investigation of Abutment Displacement of a Full Height Integral Bridges in Dense Granule Backfill

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Abstract: Problem statement: In this study, the behavior of abutment wall in full height frame integral abutment bridges was investigated. It was seen that the effect of backfill soil resistance on behavior of abutment wall movement is mostly neglected in previous studies. In this research, the final bridge superstructure displacement under temperature-induced forces was formulated. In addition, according to the final bridge displacement, the earth pressure that acts as a resistant force on the bridge abutment using the new equation from British design manual for roads and bridges, BA 42/96 was used. Besides, in the construction of integral bridges, the deck and girders are mostly encased into abutment wall, which makes these bridge components as fixed elements. This fix connectivity makes the top abutment wall move along with the bridge deck. Moreover, the abutment wall in integral bridges is made of reinforced concrete and thus, it could be assumed as a rigid mass that has a linear deformation behavior. **Approach:** To implement a new method to calculate the amount of abutment wall movement at different elevations in full height frame abutment integral bridges, considering the parameters such as temperature changes, bridge deck elongation and the backfill soil resistance. First, internal forces of the bridge abutment were formulated. They were all presented as functions of bridge deck final displacement. Second, different methods to calculate the soil lateral pressure were used. Third, the numerical modeling was applied and the corresponding results due to the bridge deck elongation were extracted. Fourth, the results obtained from phases two and three were compared to obtain some conclusion. **Results:** The results derived in this study, consisted of four data sets. First, the existing forces such as the bridge deck elongation force, the backfill soil resistance etc. were formulated according to the bridge final displacement. Then after, the static principals revealed the amount of deck final elongation. For the second set, different correlations such as British Standard, Massachusetts manual and etc. which had considered the effect of deck final displacement in their formulas were presented and with regard to the first part, the backfill reactions were obtained. For the third set, by combining the results from set one and two, different values for the deck final displacement were derived. For the next step, according to the fix connectivity of the abutment and the bridge deck, the abutment top elevation displacement was set equal to the deck final displacement. For the bottom elevation, because of the rigidity of the wall and the rotational behavior about its foundation, the displacement was set zero. Therefore, by assuming linear deformation behavior of rigid masses, the abutment deformation profile for different elevations was concluded. For the last set, the bridge computer model was built using SAP2000 and the corresponding results were collected. **Conclusion:** It was seen that, generally, except for some certain cases, all the used correlations in this study were in a close agreement either with each other or with the Finite Element data. British Standard method had the closest results to the finite element data and thus preferably it is recommended while the others not denied.

Key words: Bridge deck elongation, passive soil pressure, finite element model

INTRODUCTION

Traditional bridge structures use expansion joint systems to accommodate the change in the bridge

length induced by temperature variation. Integral abutment bridges, which do not contain expansion joints, provide an attractive alternative to traditional bridges. Jointless bridges have lower construction and

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maintenance costs. The integral connection between the abutment and the bridge girders introduces additional strains and stresses in the bridge members due to thermal expansion and contraction of the bridge superstructure. As a result, the bridge displacement induces greater forces in the abutment wall and pushes it toward or away from the backfill soil (Duncan and Arsoy, 2003). Due to design guidelines that limit the maximum thermal movement of integral bridges within the range of ± 20 mm, the importance of study of the deck length change in such these bridges could be felt significantly (BA 42/96, 2003).

Integral bridges that considered in this study were assumed to have full height frame abutments as shown in Fig. 1 (BA 42/96, 2003). The frame abutment supports the vertical loads from the bridge superstructure and acts as a retaining wall for embankment earth pressures. It is connected structurally to the deck to transfer the bending moments, shear forces and axial loads to the foundation systems. In such these frames, the wall has rotational behavior, which rotates about its foundation (BA 42/96, 2003).

Daily and seasonal temperature fluctuations cause longitudinal displacements in integral abutment bridges. Resistance to expansion and contraction of the bridge is provided by abutment backfill and the interactive substructure restraint (Civjan *et al.*, 2007). Integral abutment bridges are designed to resist all the vertical and lateral loads. The daily and seasonal temperature changes result in imposition of horizontal displacements on the continuous bridge deck, the abutments and the backfill. As the lengths of integral bridges increase, the temperature-induced displacement in the bridge components and the surrounding soil may become larger and consequently the backfill soil would be densified in a greater amount in compare with the initial conditions (Arockiasamy and Sivakumar, 2005).

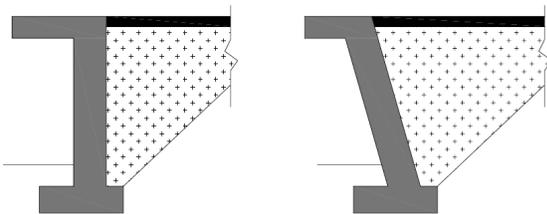


Fig. 1: Full height frame abutment in integral abutment bridges

When a bridge contracts due to decrease in temperature, the abutment wall moves away from the backfill soil. This may cause the soil, slide over the wall by loosing its lateral support. Subsequently, active earth pressure would develop behind the abutment wall (Horvath, 2000). On the other hand, when the bridge elongates due to increase in temperature, the abutment wall moves toward the backfill soil and therefore, passive earth pressure would develop behind the abutment wall (Horvath, 2008). Depending on the amount of temperature-induced displacement of abutment, earth pressure can be as low as minimum active or as high as maximum passive pressure (Arsoy *et al.*, 2004). In this study, the interaction of soil-abutment due to positive temperature changes is under investigation. Therefore, only the passive modes of abutment wall movements were considered.

MATERIALS AND METHODS

Theoretical approach: The ratio between the lateral and vertical principal effective stresses when an earth retaining structure moves away or toward the retained soil is defined as the soil lateral earth pressure coefficient. If the wall has no movement, then it would be called the at rest position and the earth pressure coefficient for this condition is defined as K_o (Budhu, 2000). There are some theories and correlations for calculation of soil lateral pressure that were proposed in the past researches. Some coefficients were defined just as functions of soil properties like in Coulomb's and Rankin's theories while in others such as British Standard, Massachusetts manual, Canadian manual and Hussein and Bagnaroil, they were proposed either as functions of soil properties or abutment wall displacement (Khodair and Hassiotis, 2005). Figure 2 shows the distributions of soil lateral coefficient and earth pressure along the abutment height (Abendroth and Greimann, 2005). The resultant soil reaction can be obtained by Eq. 1:

$$F_s = \frac{1}{8} \cdot \gamma \cdot w_e \cdot H^2 \cdot (3K^* + K_o) \quad (1)$$

In equation above:

F_s = The soil resultant force

γ = The soil bulk unit weight

w_e = The effective girders width

H = The abutment height

K_o = The soil lateral coefficient at rest

K^* = The passive soil lateral coefficient that is explained in continue

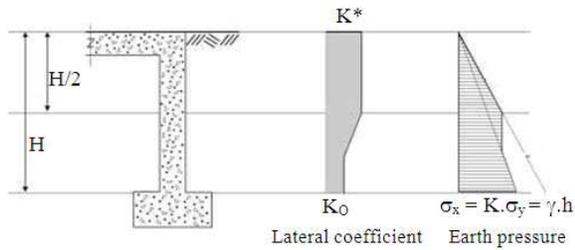


Fig. 2: Soil Lateral coefficient and earth pressure distributions over the wall height

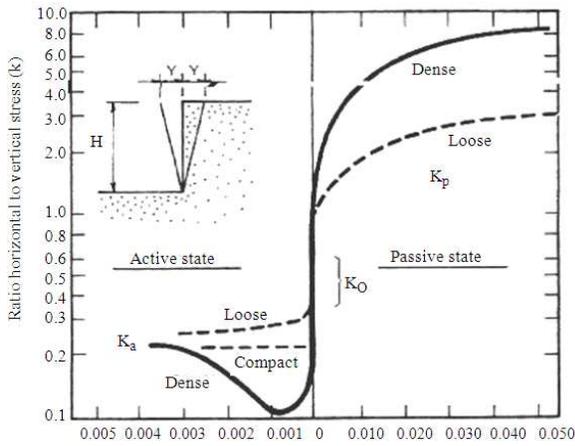


Fig. 3: Canadian manual proposed graphs for the active and passive soil lateral coefficients

As mentioned earlier, there are some correlations for calculation of soil lateral pressure coefficient. Hereby, these formulas are presented respectively. In equations below, d is the bridge deck final displacement.

British standard formula (Dicleli, 2000):

$$K^* = K_o + \left(\frac{d}{0.03H}\right)^{0.6} \cdot K_p \tag{2}$$

$$K_o = 1 - \sin \phi \tag{3}$$

$$K_p = \frac{1 + \sin \phi}{1 - \sin \phi} \tag{4}$$

where, ϕ is the soil internal friction angel.

Massachusetts manual formula:

$$K^* = 0.43 + 5.7 [1 - e^{-190.d/H}] \tag{5}$$

All parameters in Eq. 5 are as same as Eq. 2.

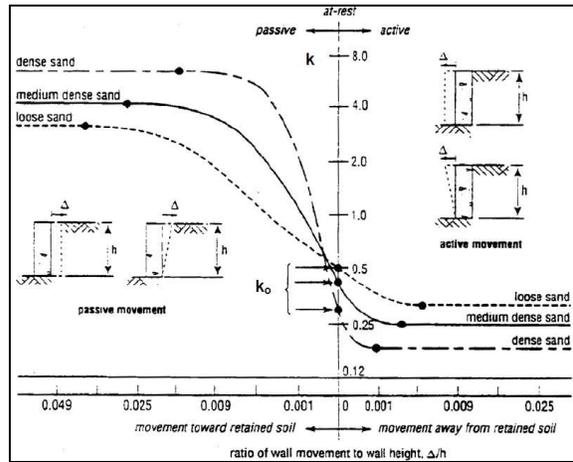


Fig. 4: Husain and Bagnaroil method for calculation of active and passive soil lateral coefficients

Canadian manual proposed formula: Canadian manual had proposed an experimental graph for the soil active and passive modes lateral coefficients. This graph is shown in Fig. 3.

For the dense sand condition ($\phi = 45^\circ$):

$$K^* = 33.26 \left(\frac{d}{H}\right)^{0.44} \tag{6}$$

Where:

- d = The bridge deck final displacement
- H = The abutment height

Husain and Bagnaroil formula: Figure 4 shows the Husain and Bagnaroil method for calculation of soil lateral coefficient.

For the dense sand condition ($\phi = 45^\circ$):

$$K^* = 10.72 \left(\frac{d}{H}\right)^{0.37} \tag{7}$$

When a bridge elongates due to increase in temperature, the backfill soil will resist by applying earth pressure on abutment wall. The intensity of earth pressure behind of the abutment is a function of magnitude of the bridge deck displacement toward the backfill soil as demonstrated in equations above and is equal to the products of soil lateral coefficients and the soil normal effective stress. As appeared in the mentioned-correlations, the magnitude of actual earth pressure coefficient, K^* , is not constant and would vary according to the amount of bridge deck movement. The soil-structure interaction model due to positive temperature changes could be best modeled as Fig. 5.

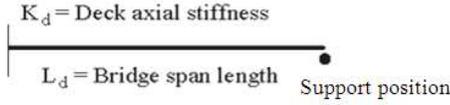


Fig. 5: Bridge superstructure structural model

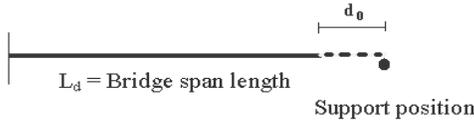


Fig. 6: Bridge deck free elongation due to temperature change, no soil resistance considered

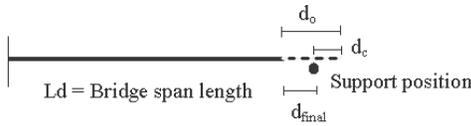


Fig. 7: Deck final elongation due to temperature change and the soil-abutment interaction

This sketch illustrated the structural model used to formulate the effect of positive temperature variation on the magnitude of earth pressure coefficient. The structural model is obtained by conservatively neglecting the resistance of piers, abutments stiffness against the structure longitudinal movement. If there was no resistance against the bridge deck elongation, the bridge deck could elongate freely under positive temperature changes. The structural model for the bridge free longitudinal displacement, d_o , due to positive temperature change is shown in Fig. 6.

The bridge free elongation is expressed by Eq. 8:

$$d_o = \frac{1}{2} \cdot \alpha \cdot L_d \cdot \Delta_T \tag{8}$$

It is clear that the soil at the back of bridge abutment would resist against deck elongation. Therefore the actual bridge deck elongation should be less than d_o . The structural model for the deck final displacement is shown in Fig. 7.

d_c is defined by Eq. 9:

$$d_c = (d_o - d_{final}) \tag{9}$$

In addition, according to the bridge final displacement, the bridge deck axial force applied to the abutment wall could be obtained by Eq. 10:

$$F_d = K_d \cdot d_c \tag{10}$$

Table 1: Bridge properties

Item	Description	Value	Unit
E_g	Girder elasticity	3.00E+10	$N\ m^{-2}$
A_g	Girder cross-sectional area	0.8169	m^2
n	Girder-slab elasticity ratio	1	-
A_s	Slab cross-sectional area in girder width	0.43884	m^2
L_d	Bridge length	96.93	m
γ	Soil bulk unit weight	18000	$N\ m^{-3}$
W_c	Girder spacing	1.8	m
H	Abutment wall height	2.56	m
α	Deck thermal coefficient	4.70E-06	1/1/F
ϕ	Soil frictional angel	45	Deg.

In equation above, K_d is the bridge axial stiffness which was defined as:

$$K_d = \frac{2 E_g (A_g + n \cdot A_s)}{L_d} \tag{11}$$

All the parameters above were defined in Table 1. By substituting the deck axial stiffness, K_d , from Eq. 11 into 10, the deck axial force could be expressed as Eq. 12:

$$F_d = \frac{2 E_g (A_g + n \cdot A_s)}{L_d} \cdot (d_o - d_{final}) \tag{12}$$

If d_o was replaced from Eq. 8, the bridge axial force could be expressed as below:

$$F_d = \frac{2 E_g (A_g + n \cdot A_s)}{L_d} \cdot (\frac{1}{2} \cdot \alpha \cdot L_d \cdot \Delta_T - d_{final}) \tag{13}$$

Assuming nearly identical abutment configurations at both sides of a bridge, the earth pressure force acting on abutment is completely transferred to the bridge deck. Therefore, to satisfy the equilibrium of forces in the longitudinal direction, the axial bridge deck force, F_d , should be equal to the earth pressure force, F_s :

$$F_d = F_s \tag{14}$$

By substituting the K^* of the soil reaction in Eq. 1 by the newly proposed formulas from British standard and the other mentioned ones, the bridge deck final displacement could be calculated. These procedures are presented below:

Deck final displacement using British standard:

$$A^* \cdot (d_{final}) + B^* \cdot (d_{final})^{0.6} - C^* = 0 \tag{15}$$

$$A^* = \frac{E_g (A_g + n \cdot A_s)}{L_d}$$

$$B^* = [E_g (A_g + n.A_s) \cdot \alpha \cdot \Delta T] - \left(\frac{1}{2}\right) \cdot \gamma \cdot w_c \cdot H^2 \cdot K_o$$

$$C^* = 3 \cdot K_p \cdot \gamma \cdot w_c \cdot H^{1.4}$$

Deck final displacement using Massachusetts:

$$A^* \cdot (d_{final}) + B^* \cdot e^{C^* \cdot (d_{final})} + D^* = 0 \tag{16}$$

$$A^* = \frac{2E_g (A_g + n.A_s)}{L_d}$$

$$B^* = -2.13 \gamma \cdot w_c \cdot H^2$$

$$C^* = \frac{-190}{H}$$

$$D^* = \frac{1}{8} \gamma \cdot w_c \cdot H^2 (K_o + 18.39) - E_g \cdot \alpha \cdot \Delta T \cdot (A_g + n.A_s)$$

Deck final displacement using Canadians:

$$A^* \cdot (d_{final}) + B^* \cdot (d_{final})^{0.44} + C^* = 0 \tag{17}$$

$$A^* = \frac{2E_g (A_g + n.A_s)}{L_d}$$

$$B^* = 12.47 \gamma \cdot w_c \cdot H^{1.56}$$

$$C^* = \frac{1}{8} K_o \cdot \gamma \cdot w_c \cdot H^2 - E_g \cdot (A_g + n.A_s) \cdot \alpha \cdot \Delta T$$

Deck final displacement using Hussein and Bagnaroil:

$$A^* \cdot (d_{final}) + B^* \cdot (d_{final})^{0.37} + C^* = 0 \tag{18}$$

$$A^* = \frac{2E_g (A_g + n.A_s)}{L_d}$$

$$B^* = 4 \gamma \cdot w_c \cdot H^{1.62}$$

$$C^* = \frac{1}{8} K_o \cdot \gamma \cdot w_c \cdot H^2 - E_g \cdot (A_g + n.A_s) \cdot \alpha \cdot \Delta T$$

Final deck displacement, d_{final} , could be obtained by solving of each equation mentioned from 15-18. As stated before, in integral bridges, the deck-abutment connections are fixed. Therefore, it could be concluded that, the deck and abutment would move as the same. This meant, the abutment wall displacement at the top elevation is equal the bridge deck final displacement.

$$\text{Abutment wall displacement}_{(Top)} = d_{final} \tag{19}$$

Further more, in full height frame abutments, the walls are rigid, which rotate about their foundations. This would lead to linear deformations of walls with zero displacement at bottom elevation:

$$\text{Abutment wall displacement}_{(Bottom)} = 0 \tag{20}$$

Figure 8 shows the abutment displacement profile along its height. The abutment wall displacement at each elevation can be obtained by Eq. 21:

$$d_i = \left(\frac{H-z}{H}\right) \cdot d_{final} \tag{21}$$

Finite element method: In order to study the bridge behavior under temperature-induced elongation, it was tried to select a model according to critical structural and geotechnical conditions. With this regard, an integral bridge with a three-span-continuous, 318-ft long, PC girder was decided to be modeled in SAP2000 computer software. This bridge had a U-shaped abutment supporting on a spread Reinforced-Concrete (RC) backwall and spread footing. A summary of the geometric characteristics of the modeled bridge is given in Table 1.

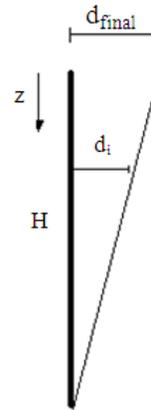


Fig. 8: Linear deformation of full height frame abutment wall

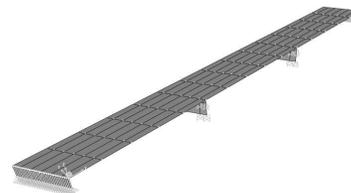


Fig. 9: Full 3-D bridge model overview built in SAP

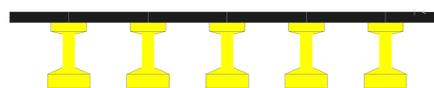


Fig. 10: Bridge slab-girders connection

Figure 9 shows the bridge overview. It was assumed that the two north and south abutment walls had identical conditions. The two intermediate piers were supported on a spread wall, which inherently produced excessive resistance against bridge deck elongation.

Figure 10 shows the bridge girders arrangement. Full-composite action was modeled between the slab and girders. Constraint equations were used to create rigid links that connected the vertically-aligned nodes of the finite elements for the slab and girders. These constraint equations coupled the translational and rotational, degrees-of-freedom between the element nodes for the slab and girders.

RESULTS

Table 2-4 present the results obtained for bridge deck final displacement. These data belongs to six categories respectively. Finite element data which took from SAP, British standard method from Eq. 15, Massachusetts method from Eq. 16, Canadian method from Eq. 17, Husain and Bagnaroil method from Eq. 18 and free bridge displacement from Eq. 8. It is important to mention that in the finite element model, the effects of existing piers were considered while in the theoretical methods, Eq. 15-18 for simplicity it was ignored.

Table 2: Deck final displacement under low temperature changes

d_{final} (mm) for $0 \leq \Delta T \leq 30$						
ΔT (F)	SAP	Brit.	Mass.	Can.	Hus.	Free
00	0.00	0.00	0.00	0.00	0.00	0.00
10	2.00	2.17	1.09	2.20	2.24	2.28
20	4.10	4.44	3.47	4.45	4.51	4.56
30	6.20	6.66	5.82	6.73	6.79	6.83

Table 3: Deck final displacement under mid temperature changes

d_{final} (mm) for $40 \leq \Delta T \leq 70$						
ΔT (F)	SAP	Brit.	Mass.	Can.	Hus.	Free
40	8.30	8.43	8.16	8.99	9.06	9.11
50	10.60	11.16	10.49	11.25	11.35	11.39
60	12.40	13.40	12.77	13.47	13.60	13.67
70	14.50	15.12	15.03	15.59	15.80	15.94

Table 4: Deck final displacement under high temperature changes

d_{final} (mm) for $80 \leq \Delta T \leq 110$						
ΔT (F)	SAP	Brit.	Mass.	Can.	Hus.	Free
80	16.48	17.90	17.47	18.08	18.10	18.22
90	18.62	20.07	19.68	20.00	20.28	20.50
100	20.76	22.02	22.02	22.35	22.59	22.78
110	23.07	24.67	24.35	24.65	24.90	25.06

Table 2 shows the bridge deck longitudinal displacement under low temperature changes from 0-30°F.

Figure 11 shows the data of Table 2 for the bridge deck longitudinal displacement under low temperature changes.

Table 3 shows the bridge deck longitudinal displacement under mid temperature changes from 40-70°F.

Figure 12 shows the data of Table 3 for the bridge deck longitudinal displacement under mid temperature changes.

Table 4 shows the bridge deck longitudinal displacement under high temperature changes from 80-110°F.

Figure 13 shows the data of Table 4 for the bridge deck longitudinal displacement under high temperature changes.

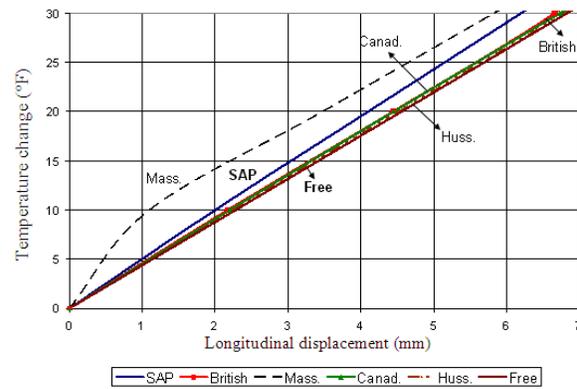


Fig. 11: Deck displacement Vs low temperature changes obtained by 6 methods

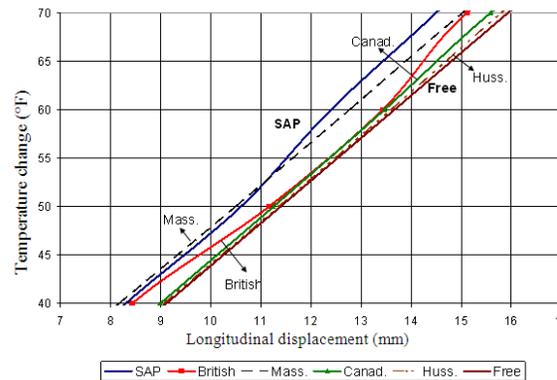


Fig. 12: Deck displacement Vs mid. temperature changes obtained by 6 methods

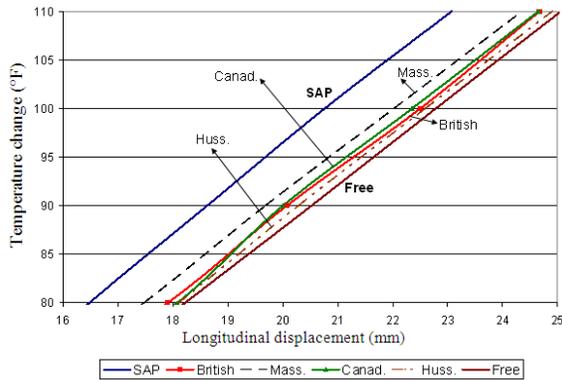


Fig. 13: Deck displacement Vs high temperature changes obtained by 6 methods

DISCUSSION

For low temperature changes as shown in Fig. 11, the results obtained by Massachusetts method underestimated the bridge deck elongation as compared with the others. In this temperature range, British Standard concluded the closest results to SAP model. The results of the other two methods, Canadian and Husain-Bagnaroil were between the range of SAP and the bridge free displacement, which showed the true integrity of the obtained results. For the intermediate temperature changes as shown in Fig. 12, Massachusetts again underestimated the bridge deck elongation up to approximately 55°F. After this temperature, its results were the closest ones to SAP. Also, the results from other methods were in the range between SAP and bridge free displacement. For high temperature changes, as shown in Fig. 13, all results were in the range between SAP and free bridge deck displacement and in this temperature changes, Massachusetts was the closest method to SAP. Finally, it is important to mention that, none of the considered results was larger than free bridge displacement and this showed that all methods could be used in the corresponding calculations.

CONCLUSION

With regard to the presented materials in this study, these below items were concluded:

- In study of abutment wall displacement, the effects of bridge deck elongation and the backfill soil on each other should be significantly considered
- As the bridge deck and abutment wall are constructed integrally in such these structures, it

could be concluded, the abutment wall movement at its top elevation is equal to the amount of deck elongation

- Abutment wall in Integral Bridges are mostly constructed in reinforced concrete, therefore it could be assumed as a rigid mass, which has a linear deformation behavior
- In full-height frame abutments, the walls rotate about their foundations. Thus, the abutment wall movement at the bottom elevation could be ignored
- Rankin and Coulomb theories may not consider the effects of deck elongation and soil resistance in their proposed formulas. Hence, they may not be proper to be used in the corresponding calculations
- In low temperature changes, Massachusetts method underestimates the deck elongation as compared to the other methods
- Approximately all the results obtained from British Standard, Massachusetts, Canadian and Husain-Bagnaroil, except for the Massachusetts low and mid temperature changes, were in the range between SAP and the deck free displacement. This can assure the integrity of these methods
- It was seen that in dense granule backfill, for the low and mid temperature changes, British Standard was the closest method to SAP as compared to the others while in high temperature changes, Massachusetts was the closest one
- It is recommended to use British Standard method for calculation of bridge deck elongation and abutment wall movement in dense granule backfill under temperature changes, while the other methods are not denied

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