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Full Height Frame Integral Bridges Abutment-Backfill Interaction in Loose Granule Backfill

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Abstract: The abutment wall behavior of full height frame integral bridges in loose granule backfill under temperature changes was investigated within this study. Since, the effect of backfill soil resistance on behavior of abutment wall movement was mostly neglected in previous researches, the abutment-backfill interaction was selected as the research study area and therefore, deriving the final abutment displacement profile was set as the research objective. In frame abutment integral bridges, the superstructure is encased into the abutment wall in which produces a fixed connectivity. This connection results in the same movement of abutment top elevation and the bridge superstructure. Furthermore, the abutment is mostly built in reinforced concrete, thus it acts as a rigid mass with a linear deformation behavior. With regard to these points and applying a new method for calculation of bridge deck displacement, the abutment deformation profile was formulated. In the new applied method, the final bridge deck displacement was expressed as a function of backfill soil resistance utilizing some available correlations due to soil behavior, such as British Standard, Massachusetts, Canadian and Husain-Bagnaroil. Fortunately, the results obtained from these correlations were in a close agreement with each other, which confirmed the integrity of applied method. Moreover, a finite element model was built in SAP2000 for this case and subsequently the outcomes were compared with the results of applied method. It was seen that both results were consistent and in most of the cases, the British Standard concluded the closets results to the finite element as compared with the others.

Key words: Bridge superstructure displacement, rigid retaining wall, numerical modeling

INTRODUCTION

Integral construction is used to avoid problems associated with bridge deck joints and reduce the construction and maintenance costs. One of the major types of construction of integral abutment bridges is a continuous jointless deck connected integrally to the abutment. The end diaphragm or the abutment is cast monolithically with the superstructure and may be directly supported on strip footing or on a single row of piles. The structural components of a typical integral bridge consist of superstructure, abutment wall, abutment foundation, abutment backfill and wing wall if necessary (England and Tsang, 2001). Due to design guidelines, that limit the maximum thermal movement of abutment within the range of ±20 mm, the importance of study of bridge abutment movement in such these bridges could be felt significantly (Anonymous, 2003). Integral Bridge that considered in this study was assumed to have full height frame abutments supported on strip footings (Anonymous, 2003). As shown in Fig. 1, the frame abutment supports

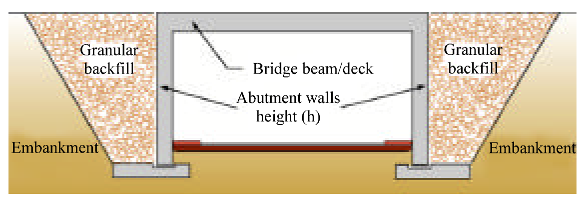

Fig. 1: Frame abutment integral bridge (England and Tsang, 2001)

abutment supports the vertical loads from the bridge superstructure and acts as a retaining wall for embankment earth pressures. In addition, the frame abutment is connected structurally to the deck to transfer the bending moments, shear forces and axial loads to the foundation system. Moreover, the frame abutment walls rotate about their foundations and have no translation at the bottom (Anonymous, 2003).

Daily and seasonal temperature fluctuations cause longitudinal displacements in integral abutment bridges. Resistance to expansion and contraction of a bridge is

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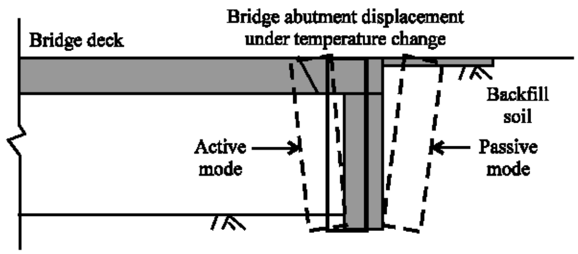

Fig. 2: Abutment wall active and passive states (Horvest, 2005)

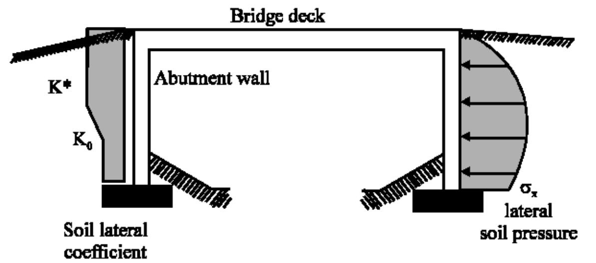

Fig. 3: Soil lateral coefficient, earth pressure distributions (Hassiotis and Xiong, 2007)

provided by abutment backfill and the interactive substructure restraint (Civjan *et al*., 2007). As the length of integral bridge increases, the temperature-induced displacement in bridge components and surrounding soil may become larger and consequently the backfill soil would be densified in greater amount as compared to initial conditions (Arockiasamy and Sivakumar, 2005). When a bridge contracts due to decrease in temperature, the abutment wall moves away from the backfill soil. This may cause the soil loose its lateral support, subsequently slide over the wall and apply an active earth pressure 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 hence, a passive earth pressure would be developed behind the abutment wall (Horvath, 2000). Depending on the amount of temperature-induced displacement of abutment, as shown in Fig. 2, earth pressure can be as low as minimum active or as high as maximum passive pressures (Arsoy *et al*., 1999). 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

This research was conducted since 2006 and the latest available correlations and theories were deployed. The research methodology consisted of three phases:

- Former formula citation
- Theoretical approach
- Numerical modeling

For the first phase, the previous formulas and correlations assessing the soil behavior due to lateral pressure were represented. In this phase, the strengths, advantages, weaknesses and deficiencies of these correlations were explained. For the second phase, the new method of calculation for the abutment wall displacement profile according to the bridge deck elongation and the soil lateral resistance was explained. In this phase, those correlations cited in phase one were applied. For the final phase, numerical modeling, a Finite Element (FE) model was deployed and subsequently the corresponding structural and geotechnical bridge components were built in SAP 2000. The aim of running FE model was to verify the integrity of the obtained results from the previous phases. Also to investigate whether, the deployed method is in a close agreement with the numerical data or not. In continue, each phase is explained in details consecutively.

Formula citation: 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 symbol for this state is K_0 (Budhu, 2007). 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 Husain-Bagnaroil, they were proposed either as functions of soil properties or abutment wall displacement (Anonymous, 2003). Figure 3 shows the distributions of the soil lateral coefficient and the earth pressure along the abutment height.

According to Fig. 3, Eq. 1 expresses the resultant force applied on the back of the wall.

$$F_s = \frac{1}{8}\gamma w_e H^2 (3K^* + K_0) \tag{1}$$

In Eq. 1, F_s is the soil resultant force, γ is the soil bulk unit weight, W_e is the effective girders width, H is the abutment height, K_0 is the at rest soil lateral coefficient and K* is the passive soil lateral coefficient that is explained in continue.

As mentioned earlier, there are some correlations for calculation of soil lateral pressure coefficient. Some of them are presented below, respectively. In these equations, d is the bridge deck final displacement and H is the abutment wall height.

British standard formula (Anonymous, 2003):

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

While,

$$K_o = 1 - \sin\phi \quad \text{(Budhu, 2007)} \tag{3}$$

$$K_p = \frac{1+\sin\phi}{1-\sin\phi} \quad \text{(Budhu, 2007)} \tag{4}$$

where, ϕ is the soil internal friction angel that was assumed as 30° for the loose granule backfill.

Massachusetts manual formula (Abendroth and Greimann, 2005):

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

All the parameters are as same as Eq. 2.

Canadian manual proposed formula:

$$K^* = 8.26\,\left(\frac{d}{H}\right)^{0.32} \tag{6}$$

All the parameters above are like Eq. 2.

Husain-Bagnaroil formula: (Abendroth and Greimann, 2005):

$$K^* = 19.17\,\left(\frac{d}{H}\right)^{0.27} \tag{7}$$

All the parameters above are similar to Eq. 2.

Theoretical approach: 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 coefficient 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. 4 (Dicleli, 2000).

Figure 4 shows the structural model used to formulate the effect of positive temperature variation on 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. 5 (Dicleli, 2000).

In Fig. 5, d_o is the free bridge elongation assuming no constrain against bridge expansion. The bridge free elongation is expressed by Eq. 8:

$$d_o = \tfrac{1}{2}.\alpha.L_d.\Delta_T \tag{8}$$

The fact is 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. 6 (Dicleli, 2000).

In Fig. 6, d_o is the bridge deck free elongation, d_c is the amount of deck contraction due to backfill resistance and d_{final} is the final position of bridge deck due to temperature-induced expansion force. The bridge deck contraction, 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, F_d, applied on the abutment wall could be obtained by equation below:

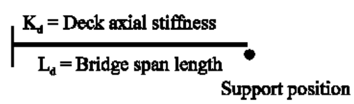

Fig. 4: Bridge superstructure structural model

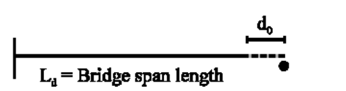

Fig. 5: Bridge deck free elongation due to temperature change, no soil resistance considered

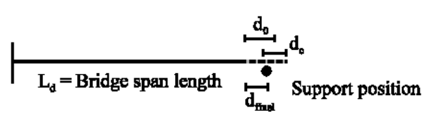

Fig. 6: Deck final elongation due to temperature change and the soil-abutment interaction

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

In Eq. 10, K_d is the bridge axial stiffness which was defined as:

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

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

$$F_d = \frac{2\,E_g(A_g + n\,A_s)}{L_d}.(d_0 - d_{final}) \tag{12}$$

If d_0 was replaced from Eq. 8, the bridge axial force could be expressed as below:

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

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* in Eq. 1 with the mentioned formulas presented in Eq. 2, 5, 6 and 7, the bridge deck final displacement could be calculated. These procedures are presented below:

Deck final displacement using British Standard:

$$\begin{aligned} & A^*(d_{final}) + B^*\,(d_{final})^{0.6} - C^* = 0 \\ & A^* = \frac{E_g(A_g + n.A_s)}{L_d} \\ & B^* = [E_g(A_g + n\,A_s).\alpha.\Delta_T] - (1/2\,\gamma w_e H^2 K_o) \\ & C^* = 3\,K_p\,\gamma w_e H^{1.4} \end{aligned} \tag{15}$$

Deck final displacement using Massachusetts:

$$\begin{aligned} & A^*(d_{final}) + B^*\,e^{C^*.(d_{final})} + D^* = 0 \\ & A^* = \frac{2E_g(A_g + n\,A_s)}{L_d} \\ & B^* = -2.13\gamma w_e H^2 \\ & C^* = \frac{-190}{H} \\ & D^* = \frac{1}{8}\gamma w_e H^2(K_o + 18.39) - E_g\,\alpha\Delta T(A_g + n\,A_s) \end{aligned} \tag{16}$$

Deck final displacement using Canadians:

$$\begin{aligned} & A^*(d_{final}) + B^*(d_{final})^{0.32} + C^* = 0 \\ & A^* = \frac{2E_g(A_g + nA_s)}{L_d} \\ & B^* = 3.09\,\gamma.w_e.H^{1.67} \\ & C^* = \frac{1}{8}K_o\gamma w_e H^2 - E_g(A_g + n\,A_s).\alpha.\Delta T \end{aligned} \tag{17}$$

Deck final displacement using Husain and Bagnaroil:

$$\begin{aligned} & A^*(d_{final}) + B^*(d_{final})^{0.27} + C^* = 0 \\ & A^* = \frac{2E_g(A_g + n\,A_s)}{L_d} \\ & B^* = 7.18\,\gamma w_e H^{1.72} \\ & C^* = \frac{1}{8}K_o\gamma w_e H^2 - E_g(A_g + n\,A_s).\alpha.\Delta T \end{aligned} \tag{18}$$

Final deck displacement, d_{final}, could be obtained by solving each equation from Eq. 15 to 18. As stated before, in integral bridges, the connection of deck and abutment is fixed. Therefore, the deck and abutment would move in the same direction and the same magnitudes. It means, 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 level.

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

Figure 7 shows the abutment displacement profile along its height. The abutment wall displacement at each elevation can be obtained by Eq. 21.

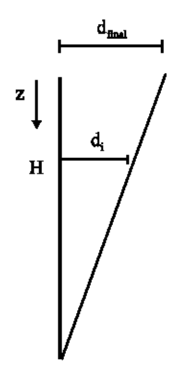

Fig. 7: Linear deformation of full height frame abutment wall

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

Numerical modeling: In order to study the bridge behavior under temperature-induced elongation, a model according to critical structural and geotechnical conditions was selected. With this regard, an integral bridge with a three-span-continuous, 318 ft long, PC girder was modeled in SAP 2000 computer software. This bridge had a U-shaped frame abutment supporting on a spread Reinforced-Concrete (RC) backwall with strip footing. A summary of the geometric characteristics of the modeled bridge is shown in Table 1.

Figure 8 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 strip walls, which inherently produced excessive resistance against bridge deck elongation.

Figure 9 shows the bridge girders arrangement. Full-composite action was assigned between the slab and girders. Constraint equations were used to create rigid links to connect the vertically-aligned nodes of finite elements for the slab and girders. These constraint equations coupled the translational and rotational, degrees-of-freedom between nodes of slab and girders.

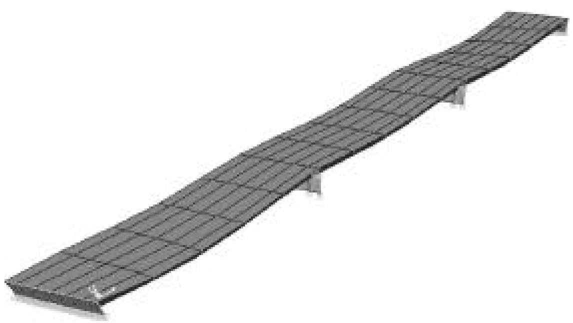

Fig. 8: Full 3-D bridge model overview built in SAP

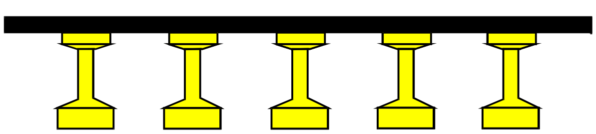

Fig. 9: Bridge slab-girders connection

Table 1: Bridge properties

Items	Description	Values	Units
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_e	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	30	Deg.

RESULTS

Table 2 shows the values obtained for equations multipliers. These multipliers are obtained from British standard manual expressed in Eq. 15, Massachussetts manual expressed in Eq. 16, Canadian manual in Eq. 17 and Husain-bagnaroil method in Eq. 18. For all the methods, the A* multipliers were 7.77E+08, the B* multiplier for British Standard was 1.09E+06, for massachusetts was -4.52E+05, for Canadaians was 4.82E+05 and for Husain-Bagnaroil was 1.17E+06. The C* multiplier for Massachusetts was -7.42E+01. The other mutilpiers were shown in Table 2. To obtain the bridge deck final displacement, the multipliers values were substituted from Table 2 into Eq. 15 to 18 consecutively again. Afterward, the derived d_{final} values were classified due to low, mid and high temperature change ranges in Table 3-5, respectively.

Table 3 shows the bridge deck longitudinal displacements for the low-range temperature changes varies from 0 to 30°F.

Figure 10 shows the data of Table 3 in the columns pattern.

Table 2: Equation 15 to 18 multipliers

ΔT (F)	Brit.	Mass.	Can.	Hus.
	C*			
10	-1.72E+06	-1.27E+06	-1.76E+06	-1.76E+06
20	-3.49E+06	-3.04E+06	-3.53E+06	-3.53E+06
30	-5.26E+06	-4.81E+06	-5.30E+06	-5.30E+06
40	-7.03E+06	-6.58E+06	-7.07E+06	-7.07E+06
50	-8.80E+06	-8.35E+06	-8.84E+06	-8.84E+06
60	-1.06E+07	-1.01E+07	-1.06E+07	-1.06E+07
70	-1.23E+07	-1.19E+07	-1.24E+07	-1.24E+07
80	-1.41E+07	-1.37E+07	-1.42E+07	-1.42E+07
90	-1.59E+07	-1.54E+07	-1.59E+07	-1.59E+07
100	-1.77E+07	-1.72E+07	-1.77E+07	-1.77E+07
110	-1.94E+07	-1.90E+07	-1.95E+07	-1.95E+07

Table 3: Deck final displacement under low temperature increase

	d_{final} (mm) for $0 \le \Delta T \le 30$					
ΔT (F)	SAP	Brit.	Mass.	Can.	Hus.	Free
0	0	0	0	0	0	0
10	2.00	2.18	1.10	2.20	2.24	2.28
20	4.10	4.44	3.48	4.45	4.51	4.56
30	6.20	6.70	5.83	6.73	6.79	6.83

Table 4: Deck final displacement under mid temperature increase

	d_{final} (mm) for $40 \le \Delta T \le 70$					
ΔT (F)	SAP	Brit.	Mass.	Can.	Hus.	Free
40	8.30	8.96	8.16	8.99	9.06	9.11
50	10.60	11.23	10.50	11.25	11.35	11.39
60	12.80	13.53	12.80	13.47	13.60	13.67
70	15.10	15.71	15.10	15.78	15.91	15.94

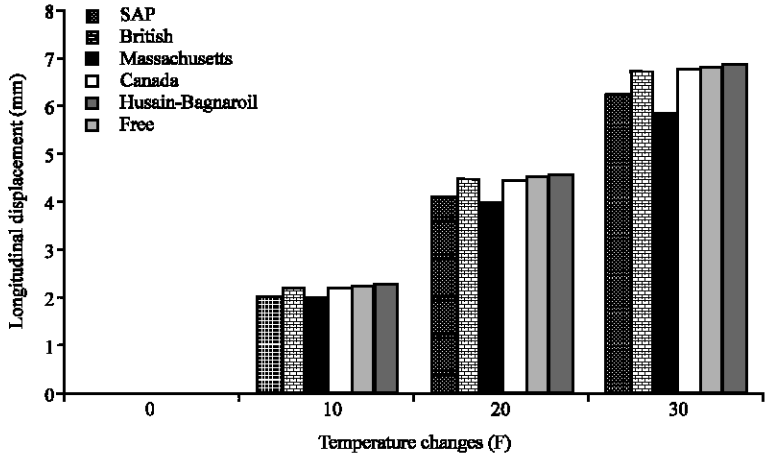

Fig. 10: Deck longitudinal displacement (d_{final}) vs. temperature changes (low-range)

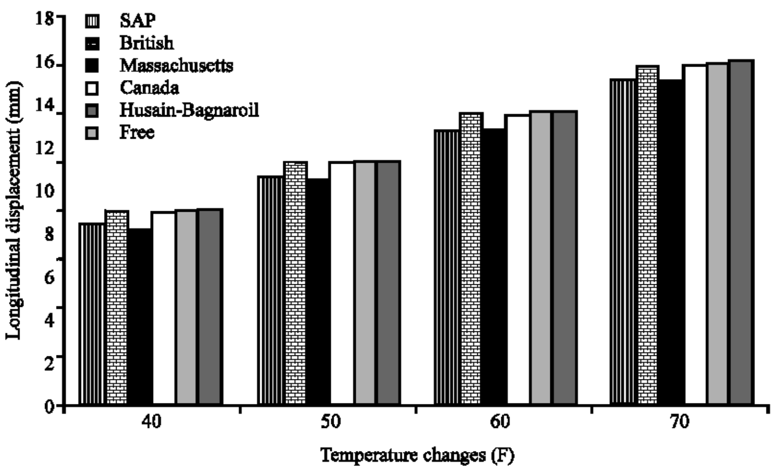

Fig. 11: Deck longitudinal displacement (d_{final}) vs. temperature changes (mid- range)

Table 5: Deck final displacement under high temperature increase

	d_{final} (mm) for $80 \le \Delta T \le 110$					
ΔT (F)	SAP	Brit.	Mass.	Can.	Hus.	Free
80	17.30	18.02	17.50	18.08	18.23	18.22
90	19.60	20.32	19.70	19.62	20.41	20.50
100	21.80	22.63	22.02	22.58	22.73	22.78
110	24.10	24.81	24.40	24.89	25.04	25.06

Table 4 shows the bridge deck longitudinal displacements for the mid-range temperature changes varies from 40 to 70°F.

Figure 11 shows the data of Table 4 in the columns pattern.

Table 5 shows the bridge deck longitudinal displacements for the high-range temperature changes varies from 80 to 110°F.

Figure 12 shows the data of Table 5 in the columns pattern.

DISCUSSION

By taking a close look to the obtained results, it can be seen that the SAP has the lowest rangewhile the free bridge displacement the largest sets of d_{final}. This is because, the effects of existing piers were considered in SAP and hence it had the lowest ranges of results while for the other methods the effects of piers were ignored. In addition, in all the methods, the soil ressitance was taken into account while in the free bridge displacement method it was neglected and therefore, the free bridge displacement had the largest ranges of results.

For the low-range temperature changes as shown in Fig. 10, Massachusetts method underestimated the bridge deck elongation. In this temperature range, British Standard led to the closest results to SAP model as compared with the others. For the mid-range temperature

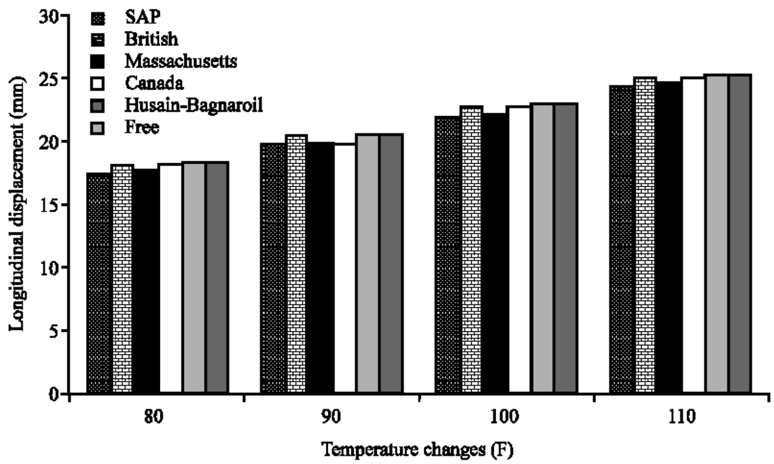

Fig. 12: Deck longitudinal displacement (d_{final}) vs. temperature changes (high-range)

changes as shown in Fig. 11, Massachusetts again underestimated the bridge deck elongation until approximately 55°F. After this temperature, Massachusetts was the closest method to SAP. For the high-range temperature changes, as shown in Fig. 12, all the results were in the range between SAP and free bridge deck displacement which this shows the integrity of the utilized methods. In this temperature range, Massachusetts method was the closest method to SAP.

CONCLUSIONS

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

- In study of abutment wall displacement, the interaction of bridge deck elongation and backfill soil should be explicitly considered
- As the bridge deck and abutment wall are constructed integrally, 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, it is 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-range 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-range and mid-range 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 loose granule backfill, for the low-range and the mid-range temperature changes, British Standard was the closets method to SAP, while in the high-range temperature changes, Massachusetts was the closets one
- It is recommended to use British Standard method for calculation of bridge deck elongation and abutment wall movement in loose granule backfill under temperature changes, while the other methods are not denied

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