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Effects of temperature changes on voided slab integral abutment bridge

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ABSTRACT Integral Abutment Bridges (IABs) are joint-less bridges whereby the deck is monolithic with the abutment walls. IABs are outperforming their non-integral counterparts in economy and safety. Thermal effects introduce significantly complex and nonlinear soil-structure interaction into the response of abutment walls and piles of the IB. This paper carried out comprehensive study on voided slab system with five spans bridge each span is 17m long. The bridge has been modelled using SAP software. The abutments and pile foundations are modeled taking into consideration the soil-structure interaction. The study covered a design uniform temperature change of (10, 20, 30, 40 and 50) °C. To gain a better understanding of the mechanism of load transfer due to thermal actions, a 3D frame analysis is carried out on the above mentioned IABs. The results showed wide range of different linear and lightly non-linear relationships between temperature range, deformations and moments. The paper highlighted the serious effect of the deformations resulting from the repeated temperature change which causes drop in soil or bombing at the abutments ~ embankment contact zone.

Keywords: Bridges, Integral Bridge, Temperature Changes, Soil-Pile Interaction.

1. INTRODUCTION

Integral Bridges (IBs) are defined as bridges construct without expansion joints or sliding bearings, thus eliminating all the issues associated with them. The continuity and single structural unit achieved by this construction with eliminating the joints increase the degree of redundancy enabling higher resistance to extreme events, such as those resulted from thermally induced deformations [1]. These in turn introduce a significantly complex and nonlinear soil structure interaction into abutment walls and piles of the Integral Bridge [1]. The unknown soil response and its effects on the stresses in the bridge, creates uncertainties in the design. The uncertainties connected with designing and performance of integral abutment bridges such as forces due to (shrinkage, creep, thermal effects (uniform temperature or temperature changes and temperature gradients), differential settlement, differential deflections, and earth pressure) can cause cracks in concrete bridge abutments [1, 2]. This paper illustrates and provides a better understanding of the mechanism of only thermal changes (temperature changes) effects without consideration of gradient temperature on one type of integral bridges decks (voided deck slab), however, for the span range of 20–30 m, it is common practice in some countries to use in-situ concrete with polystyrene 'voids' as illustrated in Fig. 1.1, which is also dependent on the type of the soil adjacent to the abutment walls and piles.

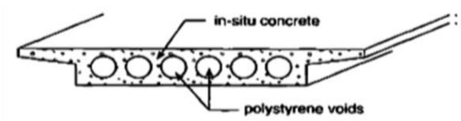


Fig. 1.1: Voided Slab Section with Cantilevers.

2. MATERIAL AND METHODS

To achieve the above objective, the following steps are performed:

- The design requirements, limitations and temperature loadings from different codes of practice were reviewed.
- Analysis and design of case study integral abutment bridge which is recently designed and constructed in Sudan.
- Perform computer mathematical model analysis for the study voided slab IAB and compare the results with the manual outcome and the as built data. Application for temperature effects will be applied.
- Perform analysis of structure for the temperature difference between 10° C and 50° C with constant interval of 10° C.
- The following codes of practice are adopted in this paper:
  - BS 5400 [9].
  - BD 37/01[12].
  - BA 42/96 [13].
- Use of integrated structural software program capable of modeling the complex and nonlinear behavior of IABs.

Comparison of analysis results is held to conclude on the voided slab IAB performance under the different ranges of temperature.

3. CASE STUDY

A.Introduction:

In 2013, the National Highway Authorities (NHA) decided to construct bridges crossing the valleys on the Karakon Hamaeshkoraib road in Eastern Sudan. Due to lack of monitoring and maintenance of the bridges in the remote rural areas, NHA selected voided deck slab integral bridge type to be constructed on these valleys. Because integral bridges have no expansion joints and no bearings, making them the most cost-effective system in terms of construction, maintenance, and longevity.

This paper performed studies on bridge 4, among a series of integral, as shown in TABLE 3.1. To draw a conclusion from this study on thermal effect on integral bridges. The design Different temperature of 50° C, 40° C, 30° C, 20° C and 10° C were used.

TABLE 3.1: Bridges on Karakon Hameshkoraib Road.

Bridge No.	At station	No. spans	Span length (m)	Total length (m)
Bridge 1	19+375	2	16	32
Bridge 2	27+027	2	16	48
Bridge 3	33+294	3	17	68
Bridge 4	41+118	4	17	85

B. Geometry of the Bridges:

Bridge 4 have an overall length of 85m between abutment center lines, with span length of 17m. The bridge carry 9.0m carriageway (2 lane), plus two 1.0m wide sidewalks and two 0.5m wide raised crash barriers. The deck slab is fully integral with the abutments and piers. The abutments and piers are reinforced concrete walls supported on pile foundation. See Figs. 3.1 to 3.3 illustrate the general views regarding Bridge 2; the other three bridges are similar to Bridge 2 except in number of spans and span lengths. The analysis and design reports [14] describe the main assumptions and output results.

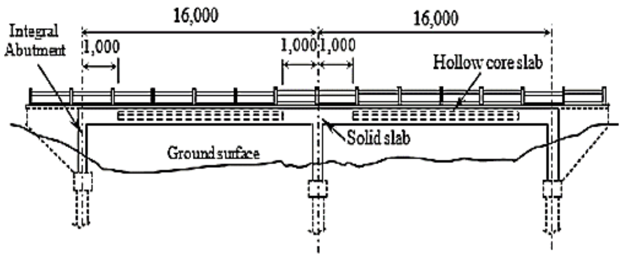


Fig. 3.1: Elevation at Bridge 2.

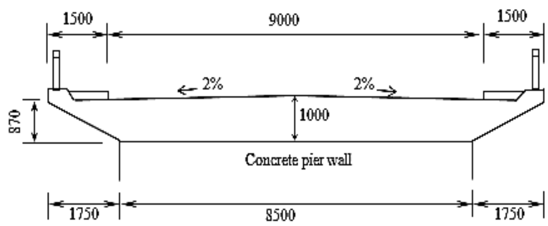


Fig. 3.2: Cross Section at Solid Part of the Deck Slab.

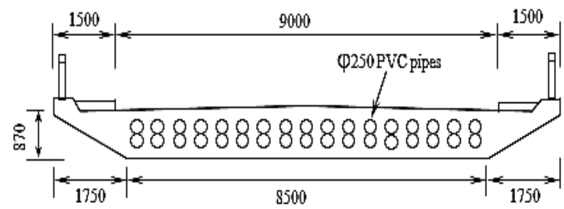


Fig. 3.3: Cross Section at Hollow Core Part of the Deck Slab.

C. Material properties:

Concrete to BS 8500:2006 [24]

Deck concrete:

C25/30 for exposure condition XD1,

f_{cu} = 30 N/mm²
E_{cs} = 28 kN/mm²
E_{cl} = 14 kN/mm²

Abutment/pier concrete:

C25/30 for exposure condition XD2,

f_{cu} = 30 N/mm²
E_{cs} = 28 kN/mm²
E_{cl} = 14 kN/mm²

Pile *concrete:*

C25/30 for exposure condition XD2,

f_{cu} = 30 N/mm²
E_{cs} = 28 kN/mm²
E_{cl} = 14 kN/mm²

D. Loading [9, 10, 11, 12]:

- *Dead load:*

Space frame method of analysis was used to analyze the bridges using SAP 2000 v17. For the purpose of grillage modeling of deck, the deck is divided into 24 sections. Fig. 3.4 shows the longitudinal grillages members labeled from section D1 to D24 at supports, and Fig. 3.6 shows the longitudinal grillages labeled from section D1 to D24 at mid span. TABLE 3.2 summarize the self -weight of the sections.

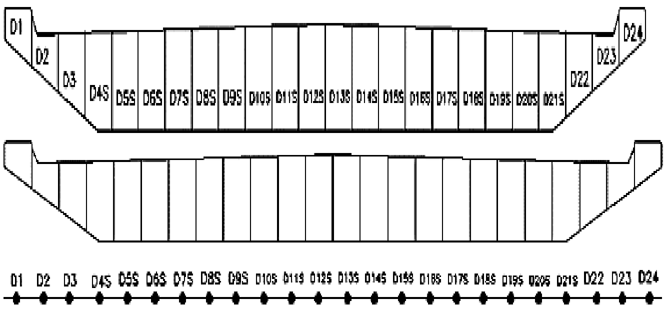


Fig. 3.5: Sections Label for Grillage Modeling at Supports.

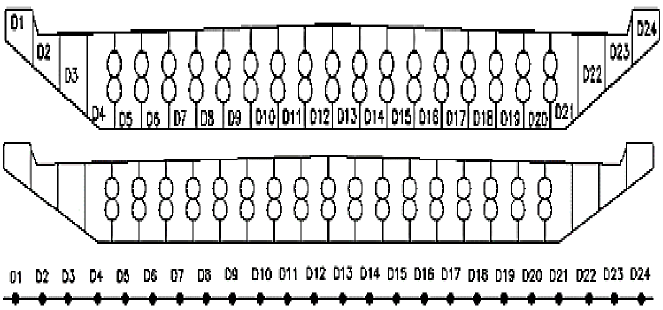


Fig. 3.6: Sections Label for Grillage Modeling at Mid-Span.

- *Uniform Temperature:*

Different design temperature is adopted (see section 3-A), and the thermal coefficient of expansion, C = 0.000012 [9, 12].

E. Soil-Structure Interaction:

- *Soil Model:*

The following soil model is obtained from Geotechnical Investigation Report.

TABLE 3.2: Summary of Sections Self Weight.

Element	X-Section Area at Mid-Span (m²)	Weight (kN/m)	X-Section Area at Support (m²)	Weight (kN/m)
D1- D24	0.2102	4.94	0.2102	4.94
D2- D23	0.2226	5.23	0.2226	5.23
D3- D22	0.3387	7.96	0.3387	7.96
D4- D21	0.3956	9.3	-	-
D4 S-D21S	-	-	0.4444	10.44
D5- D20	0.3687	8.67	-	-
D5 S- D20S	-	-	0.4663	10.96
D6- D19	0.3752	8.82	-	-
D6 S- D 19S	-	-	0.4727	11.11
D7- D18	0.3794	8.92	-	-
D7 S- D18S	-	-	0.4769	11.21
D8- D17	0.3836	9.02	-	-
D8 S- D17S	-	-	0.4811	11.31
D9- D16	0.3878	9.11	-	-
D9 S- D16S	-	-	0.4853	11.41
D10- D15	0.3917	9.21	-	-
D10 S- D15S	-	-	0.4893	11.50
D11- D14	0.3959	9.32	-	-
D11 S- D14S	-	-	0.4935	11.60
D12 – D13	0.4004	9.41	-	-
D12 S- D13S	-	-	0.4979	11.70

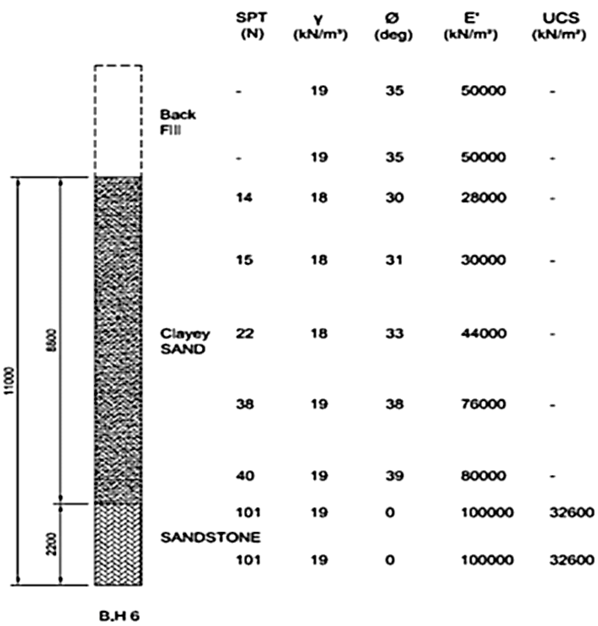


Fig. 3.7: Modeled Soil Profile Properties.

- *Soil around the piles:*

Modeled as discrete spring having a spring constant K shown in TABLE 3.3[15], which is calculated by using the following equation:

$$K_s B = 1.3\left(\frac{E_S B^4}{E_f - I_f}\right)^{\frac{1}{12}}\left(\frac{E_S}{1-\mu^2}\right) \quad (1)$$

Where:

K_S = Modulus of subgrade reaction (kN/m³).
B = Width or diameter of pile (m).
E_S = Stress- strain modulus of soil (kN/m²).

E_f = Stress- strain modulus of pile material (kN/m2).
I_f = Moment of inertia of pile (m^4).
μ = Poisson's ratio of soil (0.25 was used).

TABLE 3.3: Pile Soil Properties Used in Program Modeling.

Depth m	Unit weigh	Mod. Ø	Δ H m	q' kN/m2	α	fs	Es kN/m2	E_f kN/m2 E+07	I_f m^4	Ks kN/m
0-1.0	18	24	1	18	0.8	3.7	28000	2.80	0.049	28068
1-2.5	18	24	1.5	45	0.7	9.3	28000	2.80	0.049	28068
2.5-4.4	18	24.8	1.9	79.2	0.9	17.0	30000	2.80	0.049	30246
4.4-6	18	26.4	1.6	108	0.9	24.9	44000	2.80	0.049	45800
6-7.5	19	22.4	1.5	136.5	0.9	26.2	76000	2.80	0.049	82796
7.5-8.8	19	31.2	1.3	161.2	0.0	45.1	80000	2.80	0.049	87527
8.8-11	19	36	2.2	203	0.9	67.8	100000	2.80	0.049	111462

- *Retained soil stiffness: Hambly:* [17]

E.C. Hambly [17], suggests a range of Young's moduli of 40 to 120 MN/m 2 for material with Φ' = 35° at 10m depth. For this analysis, a mid-range value of 80 MN/m² shall be used.
For abutment with height, H = 4.0m, spring stiffness has been allocated to each spring based on blocks of soil 0.5m wide, 1.0m high.
Spring stiffness:

$$K = \frac{EA}{L} \tag{2}$$

K = (80000 × 0.5 × 1.0 × Z/10) /12 kN.m^{-1}
K at 1m = 333.33, K at 2m = 666.67, K at 3m = 1000, K at 4m = 1333.3, K at 5m = 1666.67, K at 3m = 2000, K at 4m = 2333.3 and K at 5m = 2666.67.

F. 3D Grillage Model for Voided Deck Slab Bridge:

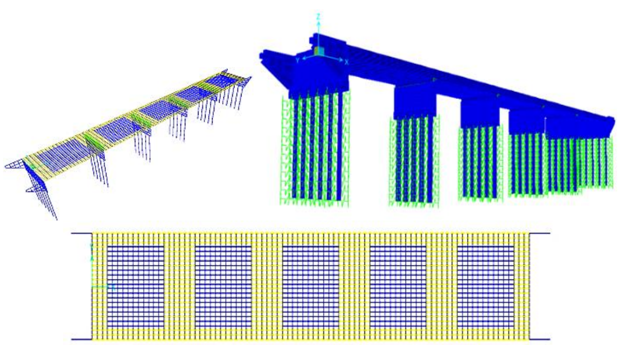
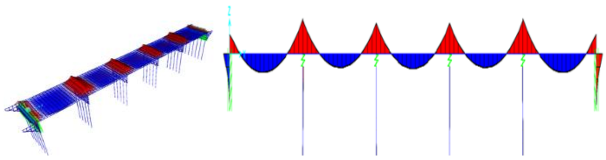


Fig. 3.8: Voided Slab 5 Spans (17m) Bridge 3D.

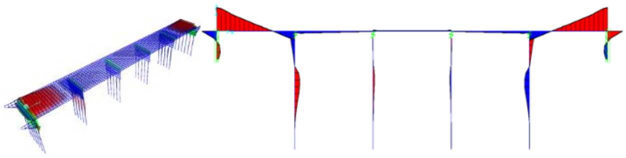
4. RESULTS AND DISCUSSIONS

A. 3D Views and Result Charts, Bending Moment Diagrams (B.M.D) Views:

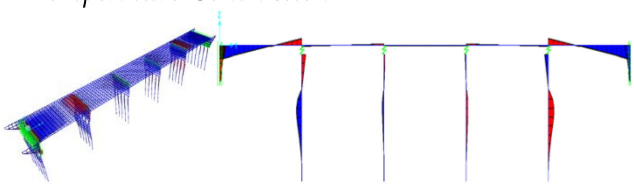
- *Dead Load:*



- *Temperature Expansion:*



- *Temperature Contraction:*



Label of Critical Sections of the Bridge:

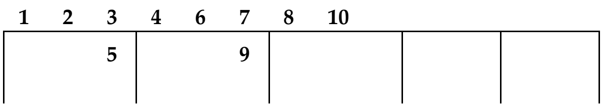


Fig. 4.1: Voided Slab, 5 (17m) Spans Bridge, (B.M.D) Views.

The following sections present the results of bending moment at points 1, 2 … 10 (see Fig. 4.1) on the bridges due to dead load and temperature effects for temperature changes of 10° C, 20° C, 30° C, 40° C and 50° C.

B. Result Charts and Discussions:

- *At Point 1:*

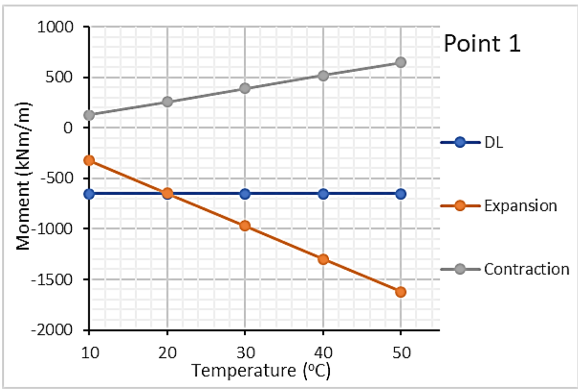


Fig. 4.2: Point 1, Result Chart due to Effects of Different Loads.

- While dead load is constant (about - 650 kN.m), the expansion and contraction temperature effects are found to be linearly proportional to temperature changes, intersecting other at 20° C, these locations have a physical interpretation regarding moments augmentation and reduction. For example, for temperature difference greater than 20° C expansion its effect is greater than the dead load effect as shown in the Fig. 4.2 above. Whilst the effect of contraction for temperature difference of 50° C it will counterbalance the dead load effect.

• The chart in Fig. 4.2 shows appreciable effects for temperature greater than 20° C.

• *At point 2:*

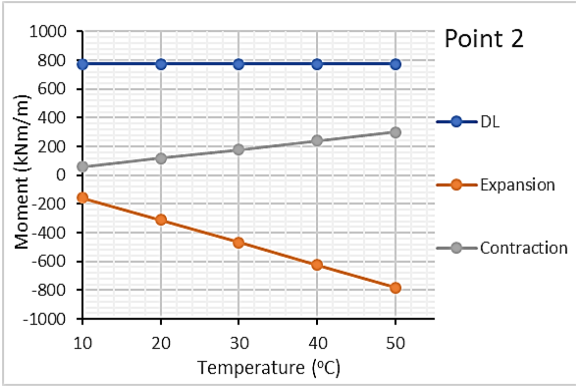


Fig. 4.3: Point 2, Result Chart due to Effects of Different Loads.

• While dead load is constant (about + 780 kN.m), the expansion and contraction temperature effects are found to be linearly proportional to temperature changes.

• At 50° C expansion effect approximately equal to the dead load effect. The above Chart Fig. 4.3 shows appreciable effects for temperature greater than 10° C while effects of temperature below 10° C may be neglected without causing significant damage.

• It is observed that the expansion is approximately 2 to 3 times of the contraction effect, this difference resulted in the restraint provided by the embankment during expansion.

• *At point 3:*

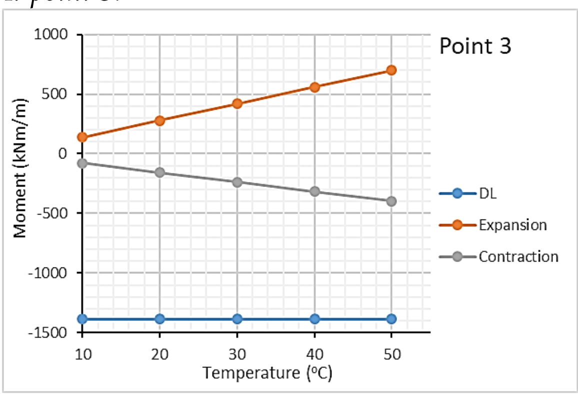


Fig. 4.4: Point 3, Result Chart due to Effects of Different Loads.

• While dead load is constant (about - 1400 kN.m), the expansion and contraction temperature effects are found to be linearly proportional to temperature changes.

• No temperature changes effect can cause moment inversion up to 50° C. Therefore, combination of dead load, contraction and reverse governs at point 3.

•The above Chart Fig. 4.4 shows appreciable effects for temperature less than 20° C effects of temperature may be neglected without causing significant damage.

• *At point 4:*

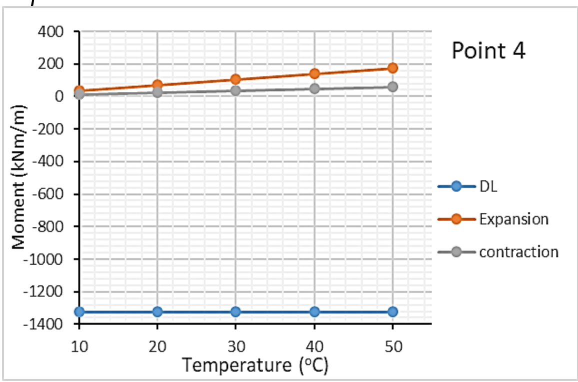


Fig. 4.5: Point 4, Result Chart due to Effects of Different Loads.

• Temperature changes effects can be neglected. Contraction have no significant effect. Therefore, the dead load (about - 1300 kN.m) governs at point 4. From 10o C to 50 o C the expansion has adverse effect to dead load, therefore, it can be ignored.

• *At point 5:*

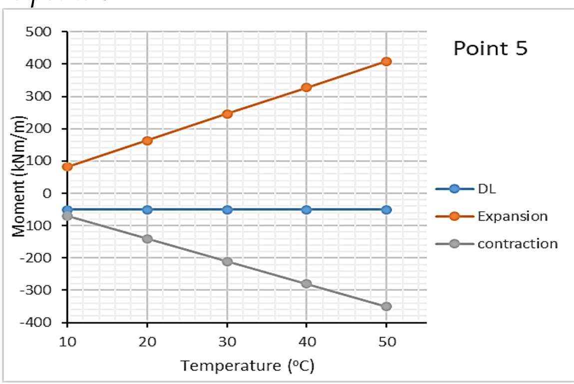


Fig. 4.6: Point 5, Result Chart due to Effects of Different Loads.

• While dead load is constant (about - 50 kN.m), the expansion and contraction temperatures are linearly proportional to temperature changes.

• Temperature effects at temperature less than 12° C can be ignored without serious consequence and the pier is designed as axially loaded compression member.

- *At point 6:*

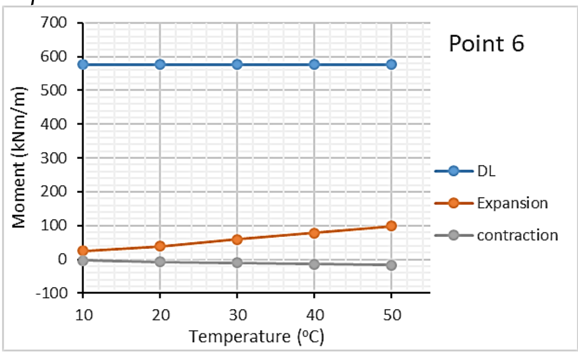


Fig. 4.7: Point 6, Result Chart due to Effects of Different Loads.

- While dead load is constant (about + 680 kN.m), the expansion and contraction temperatures are linearly proportional to temperature changes.
- Temperature effects at point 6 for temperature less than or equal to 50º C have no significant effect and can be neglected, i.e. the design is controlled by dead load only.

- *At point 7:*

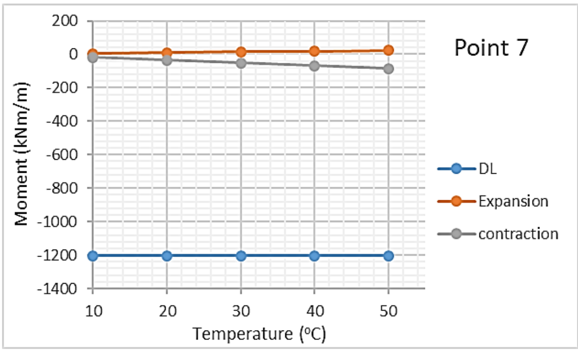


Fig. 4.8: Point 7, Result Chart due to Effects of Different Loads.

- While dead load is constant (about - 1200 kN.m), the expansion and contraction temperatures are linearly proportional to temperature changes.
- Temperature effects at point 7 for temperature less than or equal to 50º C have no significant effect and can be neglected, i.e. the design is controlled by dead load only.

- *At point 8:*

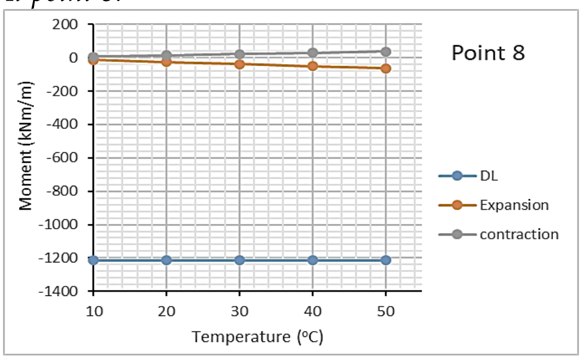


Fig. 4.9: Point 8, Result Chart due to Effects of Different Loads.

- Dead load effects are found to be constant. While the expansion and contraction temperatures are linearly proportional to temperature changes.
- Temperature effects at point 7 for temperature less than or equal to 50º C have no significant effect and can be neglected, i.e. the design is controlled by dead load only.

- *At point 9:*

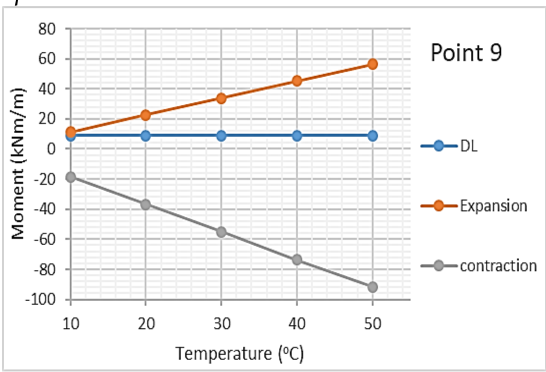


Fig. 4.10: Point 9, Result Chart due to Effects of Different Loads.

- While dead load is constant (about + 10 ≈ zero kN.m), the expansion and contraction temperatures are linearly proportional to temperature changes.
- Temperature effects at temperature up to 50º C can be ignored without serious consequence and the pier is designed as axially load compression member.

- *At point 10:*

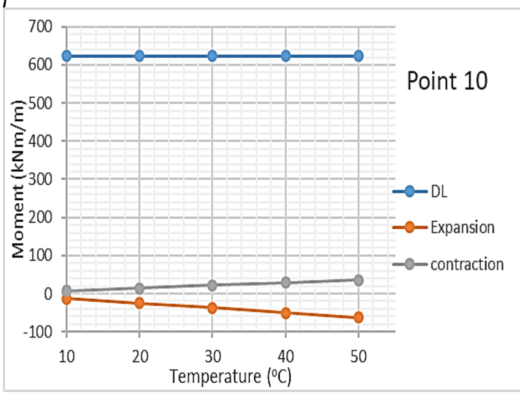
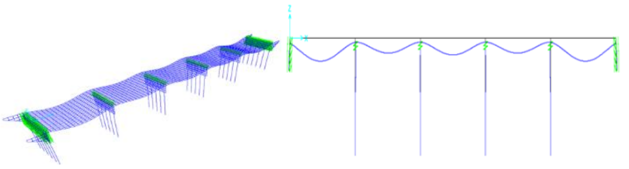


Fig. 4.11: Point 10, Result Chart due to Effects of Different Loads.

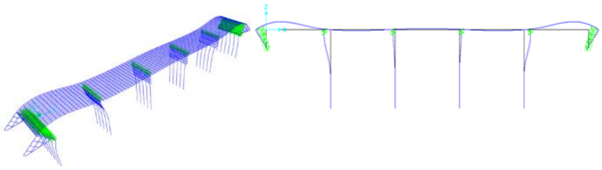
- While dead load is constant (about + 620 kN.m), the expansion and contraction temperatures are linearly proportional to temperature changes.
- Temperature effects at point 10 for temperature less than or equal to 50º C have no significant effect and can be neglected, i.e. the design is controlled by dead load only.

C.3D Views and Result Charts, Lateral Displacement:

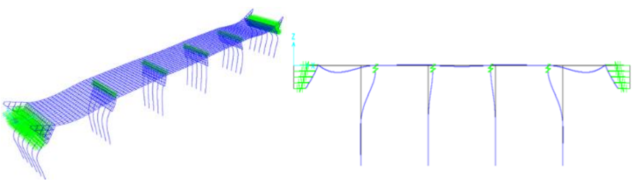
- *Dead Load:*



- *Temperature Expansion:*



- *Temperature Contraction:*



- *Point 1:*

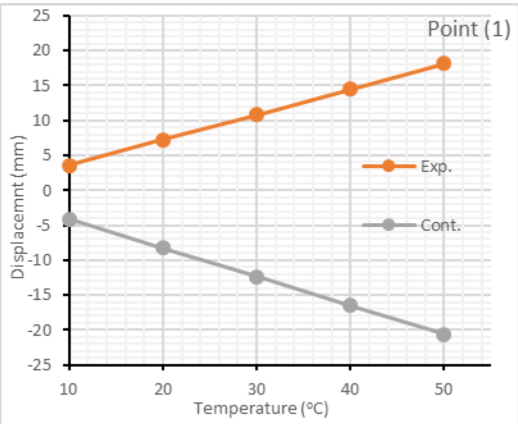


Fig. 4.12: Point 1, Result Chart Related to Effects of Different Loads.

• Contraction induces a greater lateral displacement than expansion, the difference between them is linearly proportional to temperature changes.

• Abutment is subjected to the biggest lateral displacement due to temperature effect with respect to any other point in the bridge, see Fig. 4.14 and 4.15.

• Gabbing between abutment and the adjacent soil increase from about 10mm at 10° C temperature change, with linear proportional to temperature changes, to the heist value 25mm at 10° C temperature change.

- *Point 5:*

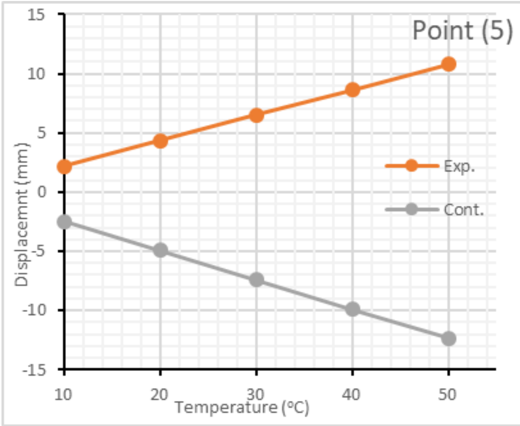


Fig. 4.13: Point 5, Result Chart Related to Effects of Different Loads.

• Contraction induces a greater lateral displacement than expansion, the difference between them is linearly proportional to temperature changes.

- *Point 9:*

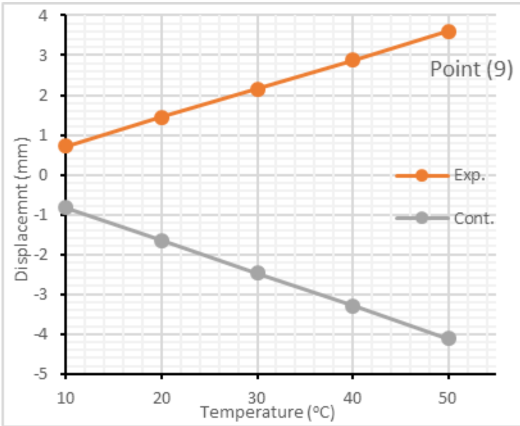


Fig. 4.14: Point 9, Result Chart Related to Effects of Different Loads.

• Contraction induces a greater lateral displacement than expansion, the difference between them is linearly proportional to temperature changes.

5. CONCLUSIONS

The conclusions drawn from this study are stated below:

• It was found that an increment of 10o C increase in temperature is adequate to produce acceptable predictions for the response of the concrete slabs subjected to temperature system of loading combined with dead loads. Live loads are not included in this study because of its transient nature.

• Integral Abutment Bridges (IABs) exposed to temperature changes equals 10, 20, 30, 40 and 50°C, the first interior span of deck has the largest deformation and moments near the abutment in all cases.

- The transverse temperature effects are of no significant effect on the stresses pattern in the structure. This is due to the absence of restraints in the transverse direction.
- Temperature effects might cause moments sign reverse (sagging to hogging and vice versa) at the top of the abutment and near mid span. This is very important in the design.
- For countries experiencing high temperature changes, like Sudan, and until further verifications are reached, the maximum total length of IAB shall be carefully controlled. Designers shall consider the effects of temperature change at temperature changes above or equal 20°C.
- In all bridge types contraction lateral displacement is greater than expansion lateral displacement along all the bridge points. This result attributed to the soil will offer no resistance to inward displacements (contraction), refer to Fig. 4.13.
- Regarding to previous point, energy dissipation in contraction is greater than that in expansion. As a consequence, to this effect, the larger moments occur due to expansion rather than contraction.
- The bending moment, shear force, and deflection in deck slab tend to increase, more or less, linearly with increase in temperature.
- Refer to Fig. 4.13 it's better to use MSE (Mechanical Stabilized Earth) when design and construct IABs to avoid gabbing problem due to the transit lateral displacements specially for the temperature change greater than 20^{o} c.

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