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Numerical investigation into thermal load responses of steel railway bridge

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Abstract. Bridge design requires consideration of the effects produced by temperature variations and the resultant thermal gradients in the structure. Temperature fluctuation leads to expansion and contraction of bridges and these movements are taken care by providing expansion joints and bearings. These movements however can be restrained by imposing certain boundary condition but at the same time considerable allowances should be made for the stresses resulting from this restrained condition since the additional deformations and stresses produced may affect the ultimate and serviceability limit states of the structure. If the reaction force generated by the restraints is very large, then its omission can lead to unsafe design. The principal objective of this research is to study the effects of temperature variation on stresses and deformation in a steel railway bridge. A numerical model, based on finite element analysis is presented for evaluating the thermal performance of the bridge. The selected bridge is analyzed and the temperature field distribution and the corresponding thermal stresses and strains are calculated using the finite element software ABAQUS. A thorough understanding of the thermal load responses of structure will result in safer and dependable design practices.
Keywords: Steel railway bridge; Temperature variations; ABAQUS; Thermal stresses and strains

1. Introduction

Uncertainties in regards to both the magnitude and consequences of thermal stresses and strains in bridges are of a [illegible]dation to design engineers. Both the short-term transient daily temperature changes and the prolonged seasonal changes cause thermal stresses and strains in the bridge. For this reason, thermal loads must be considered during bridge design and for the structural condition evaluation during the lifetime of the structure. The primary focus of this study is to enhance the comprehension of the thermal behavior of steel bridges subjected to temperature variations.

[illegible] [1] proposed a method to calculate thermal stresses and deflections in a statically determinate [illegible]n based on rigorous analyses by assuming constant longitudinal and transverse temperature, uniform temperature through the steel girder and linearly varying temperature through the cross section. [4] measured two-dimensional steady-state temperature distributions of a steel simple span bridge. To achieve a steady-state thermal condition, the top and bottom surfaces of the bridge were exposed to known thermal boundary conditions. Temperature and strain variations in the mid-span section were recorded. The finite element method was used to simulate the bridge conditions to verify its validity. [10] proposed a vertical temperature distribution for a steel girder bridge based on a synthesis of several theoretical and experimental studies on prototype bridges. [11] conducted two-dimensional heat conduction analyses by assuming constant longitudinal temperature to predict temperature distribution within steel bridges. They also reduced the two-dimensional models to single-dimensional models by further assuming constant transverse temperature. Based on finite element method, [14] developed a numerical model for the prediction of the temperature distribution of steel bridges with steel decks. The calculated temperatures were compared with the measurements taken from scaled models and good agreement was achieved.

The empirical formulae and boundary conditions of heat transfer in bridges are first outlined and then solved using a general finite element software, ABAQUS to determine the temperature distribution and the consequent thermal stresses and strains. Following that, the effects of thermal load on the representative bridge model are discussed in detail. Thereby a preliminary investigation on the temperature variation across the section of the selected representative bridge is [illegible] and the resulting thermal stresses and movements are discussed in detail. Though the track-bridge interaction is an important parameter defining the behavior of the entire structure, a detailed rail-deck interaction and dynamic response of the bridge is outside the scope of this work.

2. Methodology

2.1 Bridge description and field measurements

The numerical model is based on bridge [illegible] on the [illegible]-Mumbai line near Nagari in Maharashtra. It is a steel I-girder railway bridge spanning 13.31m in the longitudinal direction and 2.16m in the transverse direction. The total depth of the bridge is 1.212m. It is a simply supported structure resting on bearings placed on supports with the total bearing area being $1080mm^2$(332mm*325mm) on all the four edges of the bottom flanges. The values of Rayleigh damping coefficients alpha and beta are 6.[illegible]68 and 0.000407588 respectively. The assembly temperature is 30°C.

A variety of instruments were used in this study to determine some of the input variables. Thermocouples were used to measure temperature while a pyranometer was used to measure the total solar radiation on a flat surface. A wax trace box was used to measure the movements at the bearings to gain a measure of the actual boundary conditions at the support in comparison to the idealized support conditions. The temperature values measured using the thermocouples and boundary conditions determined using wax trace box were used as input variables to the numerical model.

2.2 Heat transfer mechanisms

Heat is transferred by conduction within a solid and by radiation and convection with the surrounding environment. The heat conduction can be modelled by applying the principle of Fourier's law and the [illegible] is formulated by boundary conditions.

In 1822, Fourier stated that the rate of heat transfer is proportional to the temperature gradient in a solid and established the well-known Fourier partial differential equation, which is

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial T}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial T}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial T}{\partial z}\right) + Q = c\rho \frac{\partial T}{\partial t} \quad (1)$$

where k_x, k_y and k_z are thermal conductivity values corresponding to x, y and z cartesian axes, T is the temperature at position (x, y, z) at time t and c is the coefficient of specific heat of medium.

The material is a continuum, isotropic and homogeneous. After the hydration of cement in concrete bridge and the action of welding in steel bridge, the rate of heat generation Q can be set to zero.Considering heat exchange, the boundary condition is expressed as follows:

$$k\left(\frac{\partial T}{\partial x}n_x+\frac{\partial T}{\partial y}n_y\right)+q=0 \tag{2}$$

where n_x and n_y are direction cosines of the unit outward normal vector to the boundary surface and q is rate of heat exchanged between the boundary and environment per unit area.

2.3 Temperature components

According to the different effects, the thermal load can be classified into effective temperature, vertical temperature difference and horizontal temperature difference, defined below by [illegible]. The effective temperature, which accounts for expanding and contracting of bridge components in the longitudinal direction, is the weighted mean value of temperature distributed along the cross section. The vertical temperature difference, which results in supplementary internal axial forces and bending moments in the vertical plane when the section ends are restrained, refers to the difference of temperatures between the top surface and other levels in the cross section. The horizontal temperature difference, which induces secondary internal axial forces and bending moments in horizontal plane if the deformation is constrained, represents the difference of temperature between two positions on the same level in the cross section. According to the definition, the effective temperature T_e, vertical temperature difference T_v and horizontal temperature difference T_h can be expressed as:

$$T_e=\frac{1}{A}\iiint_A T(x,y)zdxdy \tag{3}$$

$$T_v=\frac{H}{I_x}\iint_A T(x,y)ydxdy \tag{4}$$

$$T_h=\frac{W}{I_y}\iint_A T(x,y)xdxdy \tag{5}$$

2.4 Thermal strain and deformation

Thermal strains are basically of two types, i) expansion or contraction of length due to an increase or decrease in overall average temperature respectively or ii) bending of members due to the presence of a temperature gradient throughout the depth.The geometric deformations of a structure are a function of temperature. Overall increase or decrease in temperature will lead to expansion or contraction [illegible]. The thermal strain Ɛt, that develops as a structure is subjected to change in temperature ΔT, can be estimated by,

$$Ɛt=\alpha.\Delta T \tag{6}$$

where α is the coefficient of thermal expansion.

Thermal strain is usually designated as positive when it represents expansion and negative when it represents contraction. Following the definition of engineering strain, the change in length ΔL is given by,

$$\Delta L = \varepsilon_t . L = \alpha . \Delta T . L \quad (7)$$

where L is the length of the structure in the direction considered. This is the maximum change in length possible when the material is able to expand and contract freely, in the case of which no thermal stresses are produced.

A member whose ends are not restrained against rotation bend in the presence of a thermal gradient present within the member. This phenomenon is known is thermal bowing as illustrated in figure1.For simply supported structures, when the bottom portion is warmer than the top, the stresses developed will be additive to those stresses caused by live and dead loads and hence very critical.

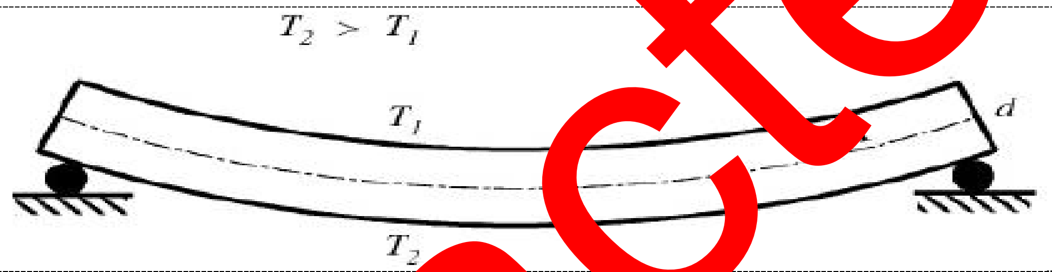

Figure 1. The phenomenon of thermal bowing

2.5 Thermal finite element analysis using ABAQUS

The partial differential equations discussed in the heat transfer section are solved using the general finite element software ABAQUS. ABAQUS has significant capabilities that are used to solve multi-physics problems. The entire bridge along with bearing was modelled and a fully coupled temperature-displacement analysis was conducted. The individual member components of the bridge namely the top and bottom flanges, web, stiffeners and the bracings are modelled as homogeneous deformable planar shell elements and then assembled to form the 3D bridge model. The shell element is considered rather than a solid element for better modelling of the thermal bowing phenomenon and to neglect the effect of temperature gradient along the thickness of the individual members owing to the negligible thickness of shell elements. The material properties of steel corresponding to both thermal analysis and the consequent stress analysis are given as input. The material properties of steel are tabulated below in Table 1.

Table 1. Material properties of steel

Material Property	Steel
Density	7850 Kg/m^3
Young's Modulus	200000 N/mm^2
Poisson's Ratio	0.3
Coefficient of Thermal Expansion	12e-6 mm/mm.K
Thermal Conductivity	43 W/m.K

A fully coupled steady state temperature-displacement analysis is executed to establish the temperature distribution field throughout the bridge and the consequent thermal stresses and strains in the structure. A good choice of time step is of vital importance, since too large of a time step may miss the peak point of interest while too small of a step leads to poor economy in computer time. The time step depends on the type of the governing partial differential equation and the features of the input. A reasonable time step was chosen for this process. Simply supported boundary conditions

(U1=U2=U3=0 at one end and U1=U2=0 at other end, where U1 is the displacement along the transverse direction, U2 is the displacement in the vertical direction and U3 is the displacement along the longitudinal direction) were assigned at the bearing areas and three different types of analysis were conducted, them being: a temperature difference of 0.2^{0}C across the depth of the bridge for temperature values 1) 27^{0}C, 2) 30^{0}C and 3) 40^{o}C. Element sizes have a significant effect on the accuracy of the results. To determine whether the element size is sufficiently fine, the number of elements is incrementally increased and comparisons are made between consequent analyses. Results obtained from the model with a certain number of elements can be compared to those obtained from the model with increased number of elements. If no significant difference is observed between them then the mesh can be deemed adequately fine. Usually the mesh convergence process involves the comparison of strain energy in the whole body with respect to the number of elements corresponding to the element size. Detailed analyses evaluating mesh sensitivity showed that an element size of 28 mm is reasonable enough for the thermal analysis. Aspect ratio is another meshing factor that can influence the accuracy of the results. Therefore, to avoid excessive distortion of the elements, the aspect ratio of elements must be limited. A 4-node thermally coupled, doubly curved, general-purpose shell element with finite membrane strains named S4T is used for this process. The meshed bridge model is presented below in figure 2.

Figure 2. Meshed model

3. Results and discussion

The following results are obtained after conducting a coupled temperature-displacement analysis on the bridge model. The various stresses and strains along the local axes are plotted below. The temperature field distribution, which is the output from the heat transfer analysis and the thermal stresses and strains, which are the significant output from the general static stress analysis are shown below. These results are then studied in detail to draw conclusions about the impact of thermal loads on the structural behaviour of the structure.

3.1 Nodal temperature-NT11

Figure 3 to 5 shows that a linear vertical temperature gradient exists along the depth of the steel bridge. This is due to the high thermal conductivity value of steel, since steel is a very good conductor of heat. In the case of concrete which has a lower thermal conductivity when compared to steel, a non-linear vertical gradient will be prevalent. The minimum and maximum temperatures recorded using the thermocouples during the field measurements and which were input as a variable field in the numerical thermal analysis, which are 27°C and 40°C respectively, is well within the specified range of -5°C to 70°C as specified by [9] and -17°C to 48°C as per [2]. According to [6], the uniform temperature component depends on the minimum and maximum temperature that the bridge is expected to experience while the vertical temperature differences are considered by using an equivalent linear temperature difference components, ΔT_{heat} and ΔT_{cool}. When the bridge is heated up, the top surface is warmer than the bottom and ΔT_{heat} is taken as 15°C. When the bridge is cooled down, the bottom surface is warmer than the top and ΔT_{cool} is taken as 18°C.

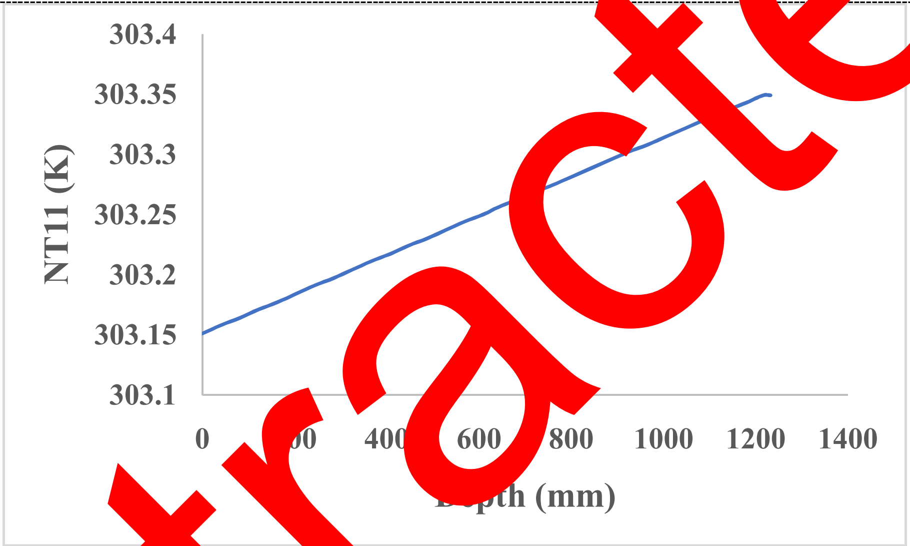

Figure 3. Linear vertical temperature gradient for case 1

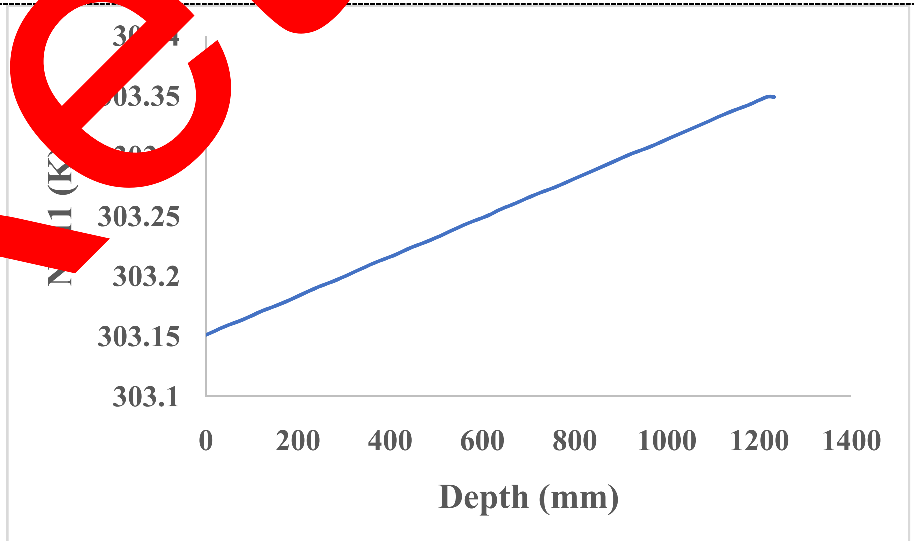

Figure 4. Linear vertical temperature gradient for case 2

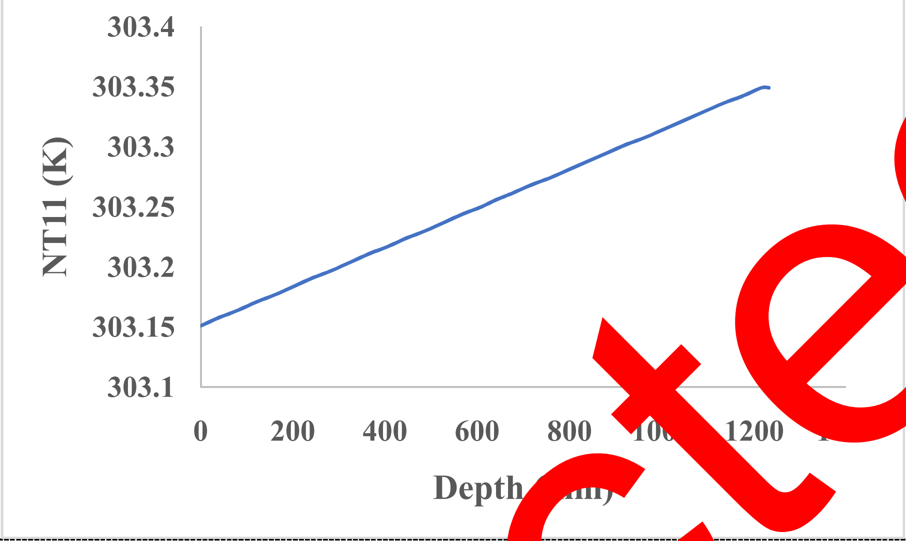

Figure 5. Linear vertical temperature gradient for case 3

3.2Normal stresses-S11&S22

From figures 6 to 8 it can be seen that the stresses vary from compression in the lower flange to tension in the upper flange, while the stresses along the web is approximately equal to zero. The tensile stresses are higher when compared to the compressive stresses since the temperature at the top flange is higher when compared to that of the bottom flange.

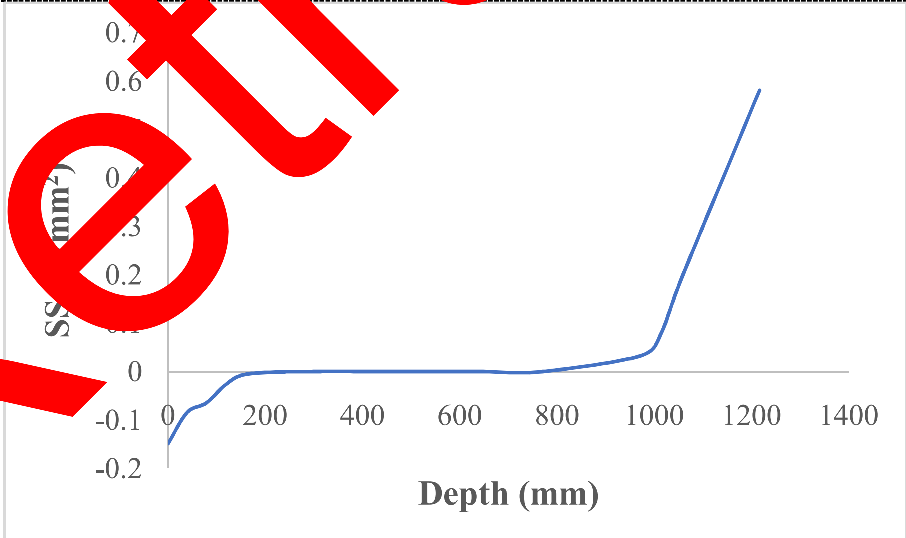

Figure 6. Variation of normal stress component S11 across the depth for case 1

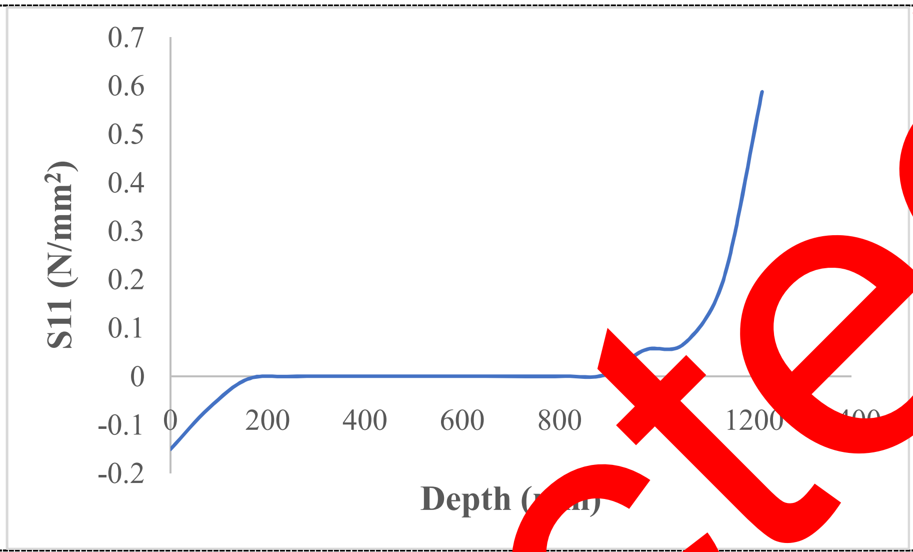

Figure 7. Variation of normal stress component S11 across the depth for case 2

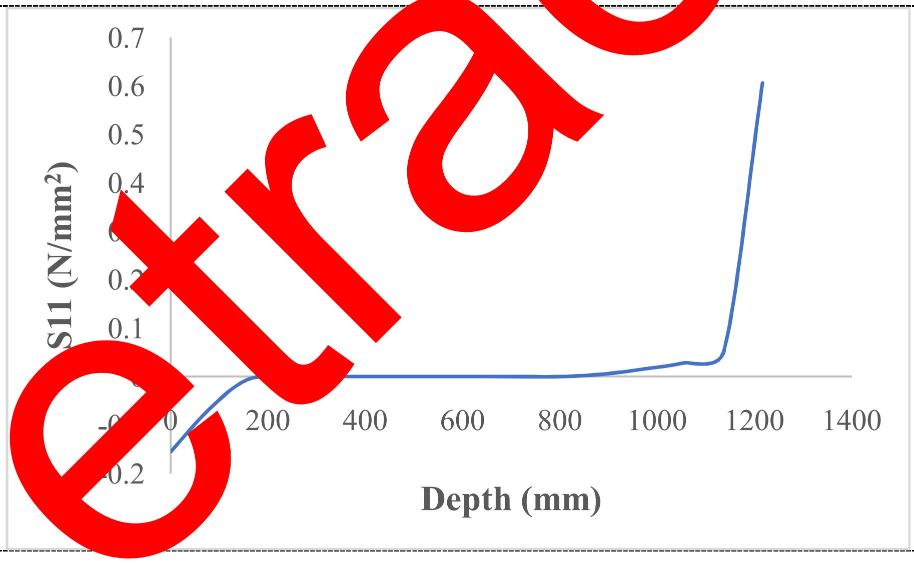

Figure 8. Variation of normal stress component S11 across the depth for case 3

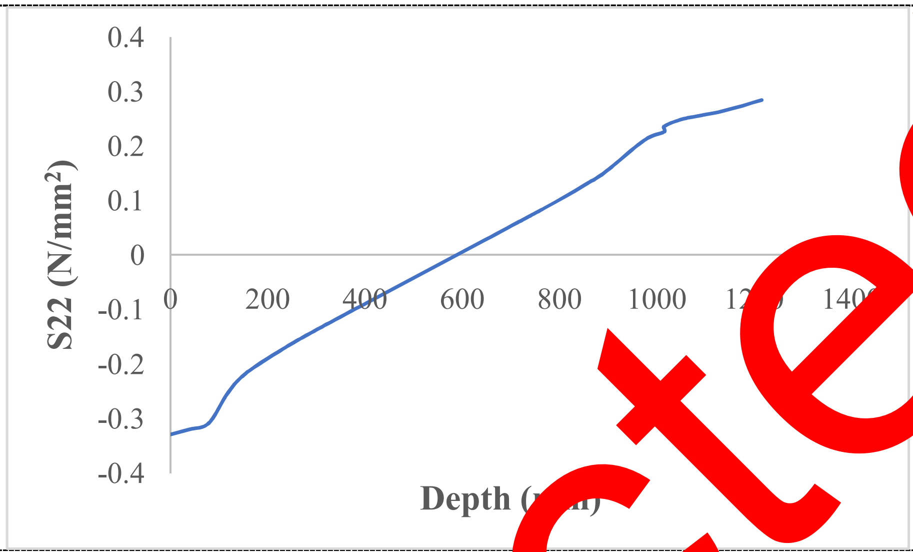

Figure 9. Variation of normal stress component S22 across the depth for case 1

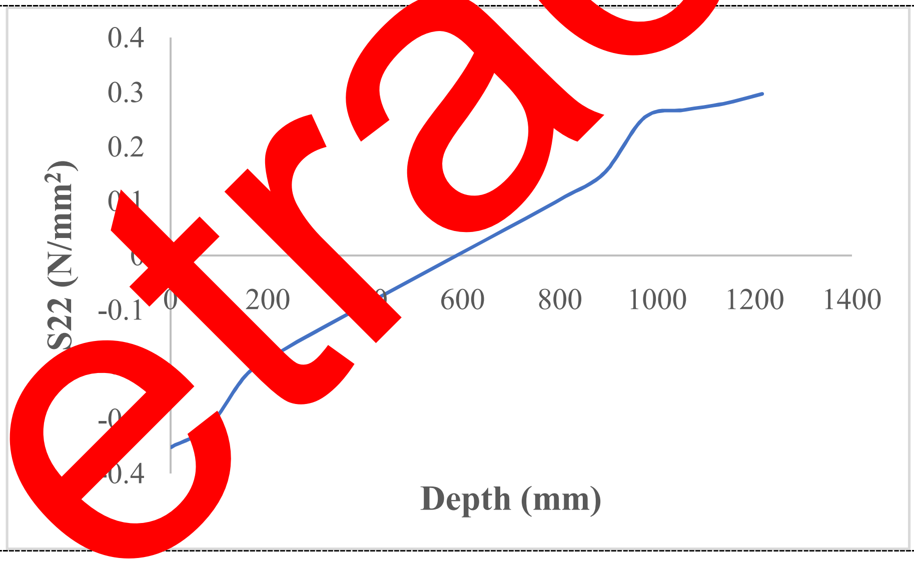

Figure 10. Variation of normal stress component S22 across the depth for case 2

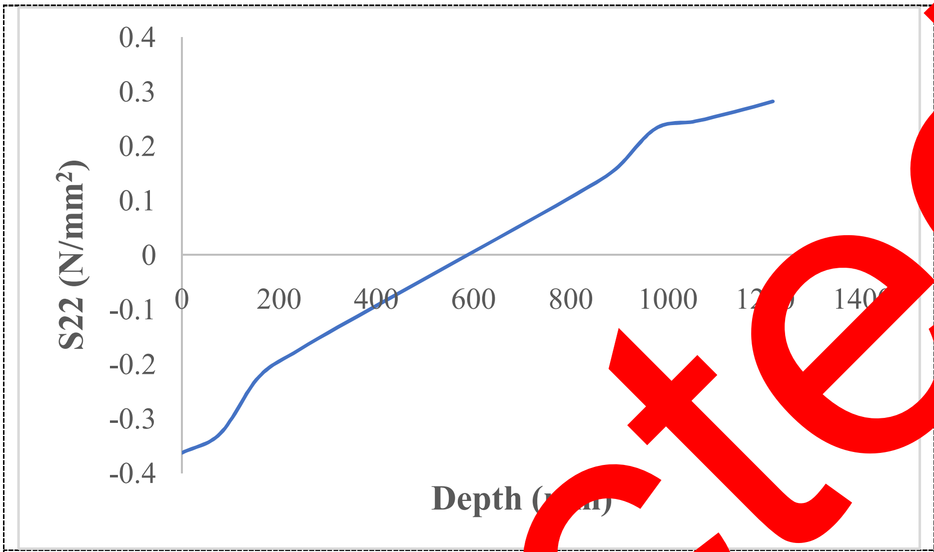

Figure 11. Variation of normal stress component S22 across the depth for case 3

A linear stress variation along the depth of the bridge as seen in figure 9 to 11 ranging from compression in lower flange to tension in upper flange. This also proves the phenomenon of thermal bowing indicating that the section arches up when the top surface is warmer than the bottom one leading to compressive stresses at the bottom and tensile stresses at the top.According to [9], design of steel railway structures usually is based on a working stress level that is some fraction of the minimum yield strength of the material. This value is usually taken as 0.55, allowing a safety factor of 1.82 against yielding of steel. Hence the working stress for structural steel with a yield strength of 250 N/mm^2 is 137.5 N/mm^2. The maximum stress obtained is 0.6 N/mm^2 experienced by the upper flange for a temperature of [illegible] K. Hence in this case thermal stresses accounts for 0.4% of the entire working stress and does not significantly affect the strength criterion of the structure. But when the field temperature is higher than the maximum value considered then there may be higher thermal stresses which may severely affect the ultimate limit states of the structure.

3.3 Shear stress-S12

Figures 12 to 14 confirms that there are shear forces at the supports or restraints giving rise to shear stresses across the bearing area. There are comparatively higher stresses at the right end which is the pin support where the displacement along all the three mutually perpendicular directions is arrested giving rise to greater stresses when compared to the left end which is the roller support which allows displacement along the longitudinal axis.

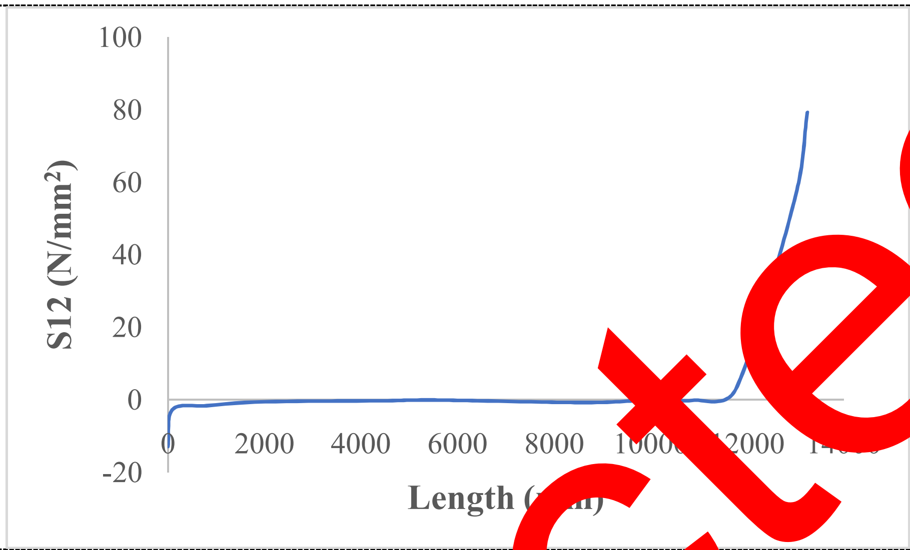

Figure 12. Variation of shear stress along the length for case 1

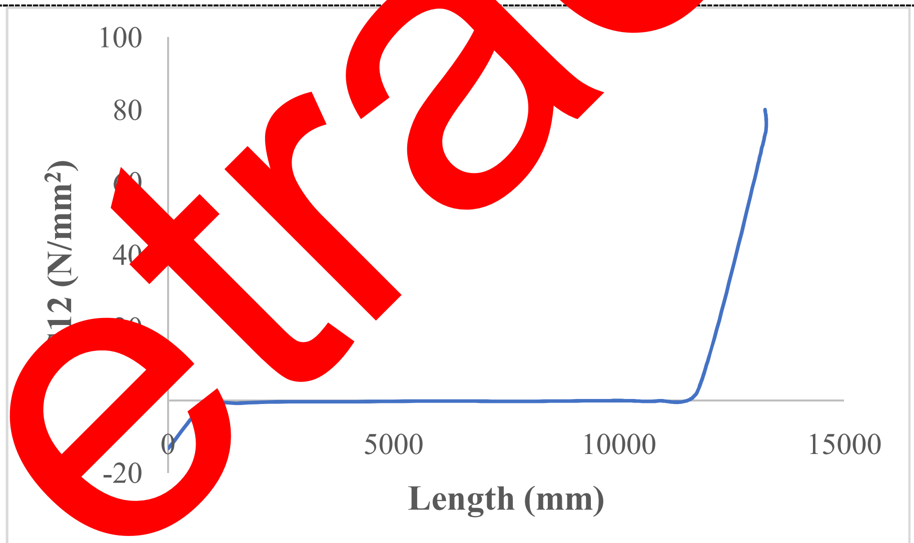

Figure 13. Variation of shear stress along the length for case 2

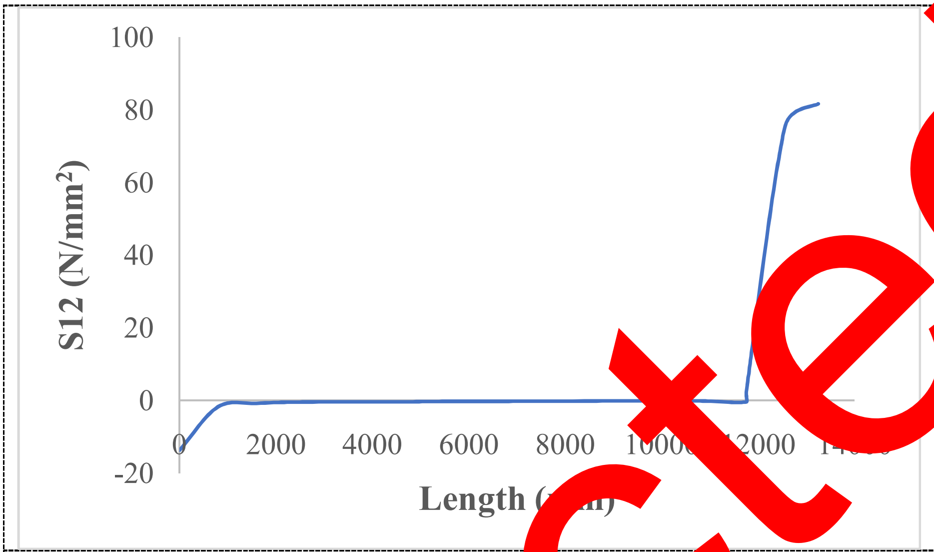

Figure 14. Variation of shear stress along the [illegible]gth for case [illegible]

3.4 Displacement along vertical direction-[illegible]

Figures 15 to 17 shows the vertical defle[illegible]on alo[illegible] th[illegible]ngth of the bridge. The deflection limit as specified by [9] should not be grea[illegible]er t[illegible]n 1/[illegible] (defl[illegible]/length of the girder). The maximum deflection obtained in our case i[illegible]4569/133[illegible]=0.000[illegible] which is less than the specified limit of 0.001. Hence the serviceability [illegible]t state of t[illegible] bridg[illegible]s not greatly affected. But in case of higher temperature, the def[illegible]ction may [illegible]eed the pe[illegible]e limit and this may influence the bridge-rail interaction which ma[illegible] vertica[illegible]splacement of the rail leading to track irregularity.

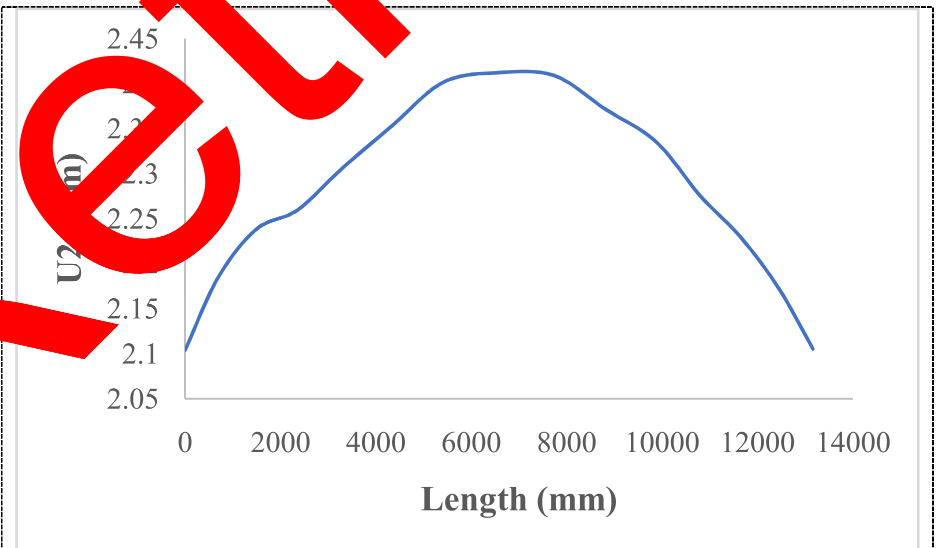

Figure 15. Deflection curves for case 1

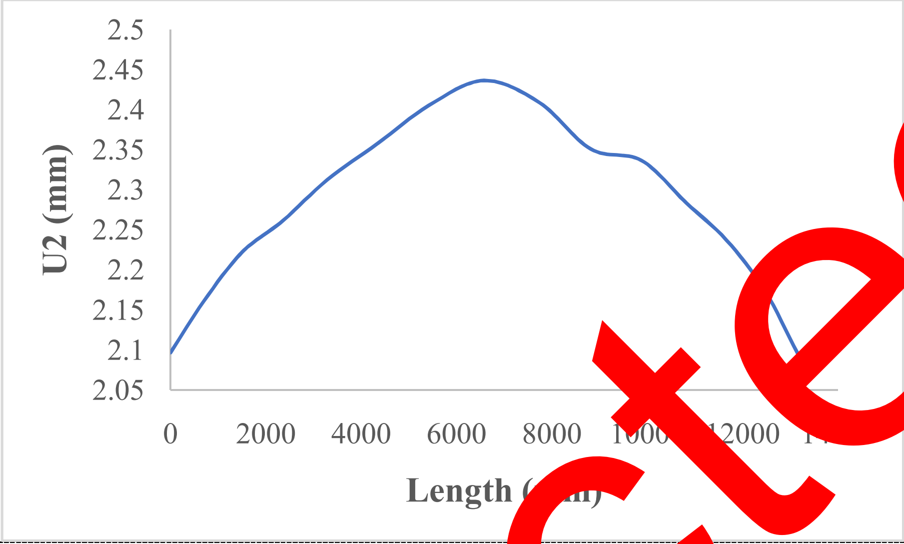

Figure 16. Deflection curves for case 2

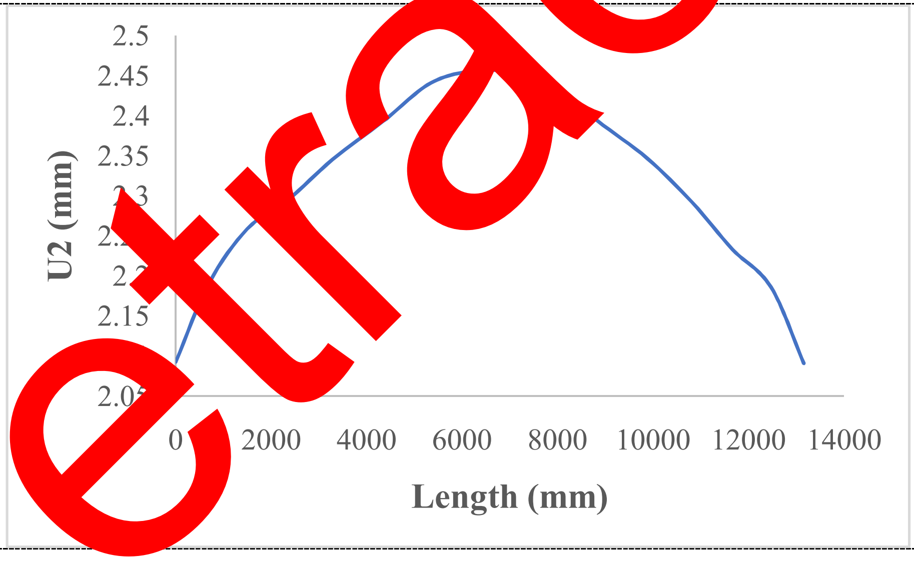

Figure 17. Deflection curves for case 3

3.5 Displacement along longitudinal direction-U3

It can be clearly noted from figures 18 to 20. That the maximum longitudinal displacement occurs at the left side roller support while there is zero displacement at the right side pinnedsupport. This movement is usually accommodated by expansion joints and bearings. According to [9], where provision for expansion and contraction, due to change in temperature and stress, is necessary, it shall be provided to the extent of not less than 25mm (1in) for every 30m (100ft) of span. The expansion bearings shall allow free movement in a longitudinal direction and at the same time prevent any transverse motion which corroborates our support conditions. [2] specifies a limit of 1 to 1¼ inches of movement for each 100 feet of span length. The maximum longitudinal displacement obtained is 49.327mm. Since this displacement is greater than the specified minimum displacement of 25mm, expansion joints should be provided at the ends of the girder.

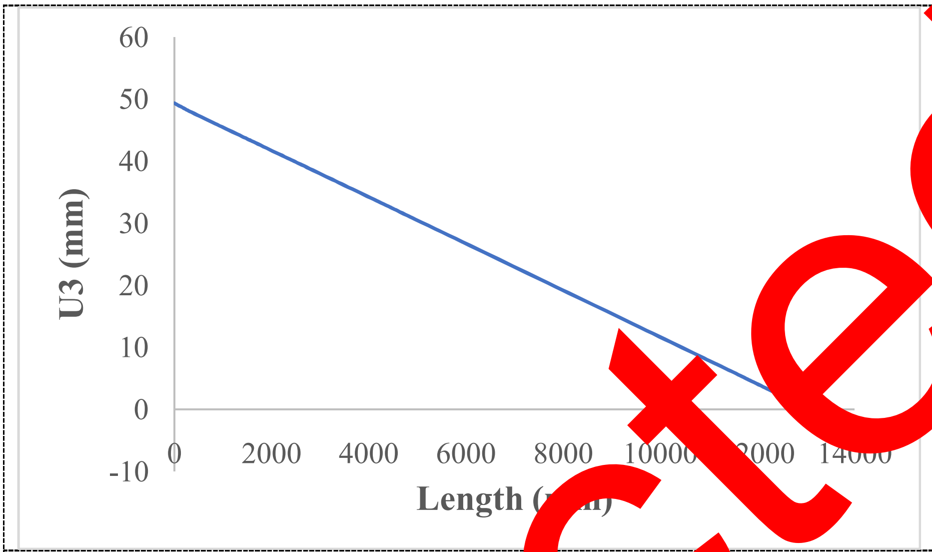

Figure 18. Longitudinal displacement for case 1

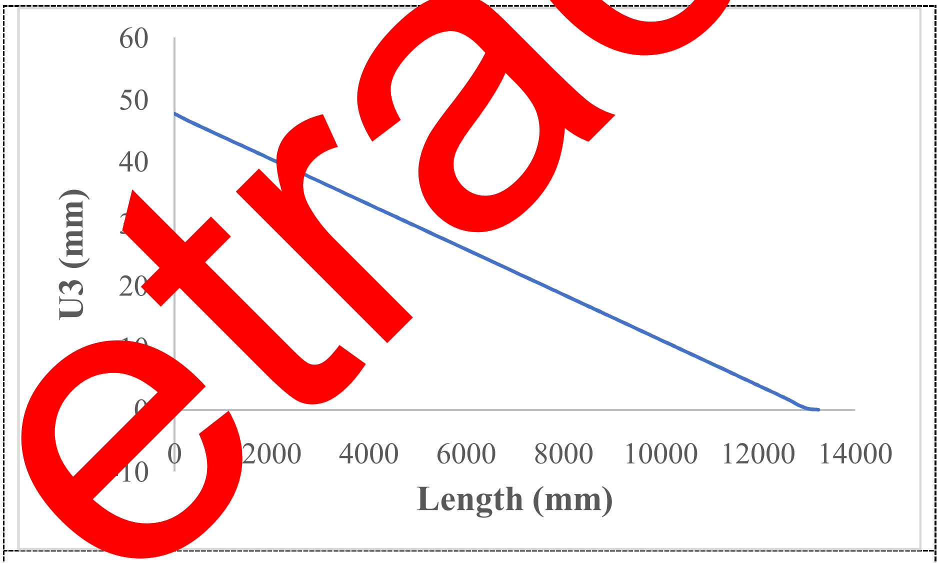

Figure 19. Longitudinal displacement for case 2

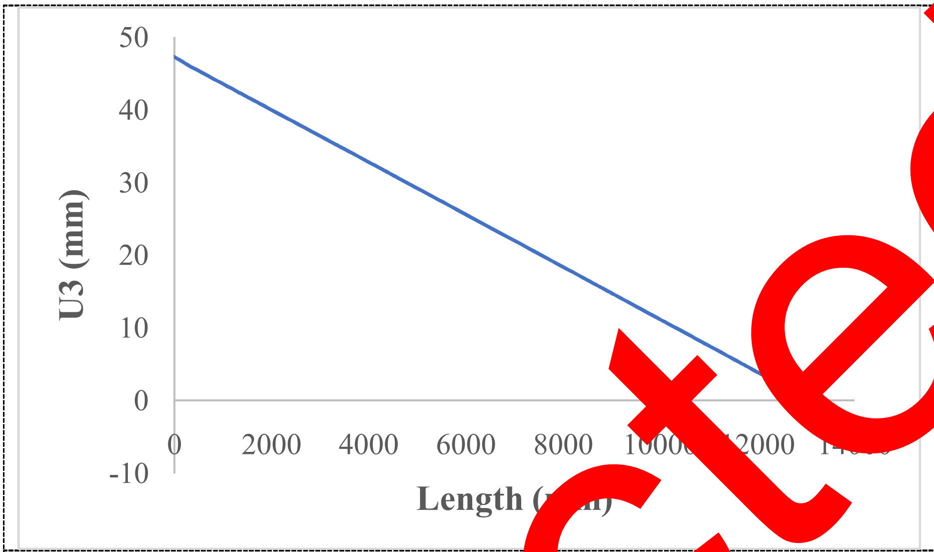

Figure 20. Longitudinal displacement for case

4. Conclusions

The main aim of this study is to demonstrate the effect of thermal gradient present within a steel railway bridge on its structural behaviour and its impact on ultimate and serviceability limit states. Due to continuous climatic fluctuations in the surrounding environment, it is inevitable that structures are constantly subjected to varying temperatures resulting in temperature gradients within them. The thermal loads depend on various factors like ambient, solar radiation, precipitation type and amount, wind effects, geographical location of the structure, orientation, material properties, geometry and so on. Thermal loads in structures result in thermal strains and in case these strains are restricted, stresses develop within the members. Hence it is of utmost importance that the thermal loads must be considered in the design in order to produce safe and reliable structures. This study includes field measurements followed by numerical analysis. A fully coupled temperature-displacement analysis is performed on the selected bridge model using the finite element software ABAQUS. After analysing the structural response of the steel bridge subjected to linear vertical temperature gradient it was concluded that if the minimum and maximum temperature the bridge is subjected to during its lifetime remains within the permissible range as specified by the codal provisions, then the thermal stresses and strains also tend to remain in the tolerable range without significantly affecting the performance of the structure. In other cases where there are extreme temperature fluctuations diligent care should be taken while designing the structure since increase in temperature leads to loss of strength and stiffness in members while varying temperatures causes varying thermal loads inducing fluctuating stresses which causes fatigue failure in members and connections. Continuous large expansion and contraction due to increase or decrease in temperatures respectively in expansion devices may lead to its early failure and repeated repair and replacement thereby increasing its maintenance cost. Numerical analysis provides a perfect approach to predict the thermal loads and the resulting thermal movements and stresses. However, these approaches are input parameters dependent and hence the predicted results and values of the various stresses and strains may deviate from the actual case since some of the input or predefined field variables are dependent on environmental factors and are subjected to changes from time to time.

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