Thermal movements of the Existing Gateway Bridge

Chris Parkinson, Geoff Taplin and John Connal, Maunsell AECOM, Melbourne, Australia

Synopsis
The Gateway Bridge, opened in 1986, is a post tensioned concrete box girder bridge crossing the Brisbane River, with a main span of 260 metres and approach spans ranging from 60 metres to 145 metres. The approach spans are twin-cell segmental box girders and the main spans are cast in-situ box girders. The approach spans have bearings at the tops and base of all piers. The twin blade piers that support the main span are integral with the superstructure.

The bridge has two internal expansion joints and is tied into each abutment, preventing longitudinal movement at these locations. On the south side, the expansion joint is 376 m from the abutment. On the north side, the expansion joint is 731 metres from the abutment. The distance between the two expansion joints, including the main span, is 520 metres. The joint consists of a needle-beam type halving joint and modular expansion joint at deck level.

The modular expansion joints on the Existing Gateway Bridge have reached the end of their useful life, and will be replaced as part of the Gateway Upgrade Project. As creep and shrinkage movements of the existing bridge have effectively ceased, the movement range of the replacement expansion joints will be largely determined by the thermal movements. In order to obtain a more accurate estimate of the thermal movement range, the movements at both of the existing modular expansion joints on the Gateway Bridge have been continuously recorded since September 2007. Ambient temperature inside the bridge has also been recorded at both expansion joint locations.

The paper summarises the thermal movements, and relates these to both the bridge ambient temperature, and the shade air temperature near the site, as recorded by the Bureau of Meteorology. The results are compared to the provisions of the Australian Bridge Design Code.

1.0 Introduction
In the Australian Bridge Design Code AS5100 (Standards Australia 2004), the calculation of extreme values of average bridge temperature is based upon the extreme values of shade air temperature. AS5100 requires that “for major or special structures, extreme shade air temperatures for the actual site shall be determined”. This requirement can be easily met for many sites around Australia, because the Bureau of Meteorology has extensive records of yearly extreme shade air temperatures at many locations. AS5100 also requires that, for “concrete superstructures .... greater than 2 m in depth, an allowance shall be made in average bridge temperatures to account for the heat sink effect.” This latter requirement is not so easy to fulfil – what allowance is appropriate, and how is it quantified?

Analytical methods are available, through two dimensional heat flow analysis, to calculate the ‘heat sink’ effect on a given concrete cross section, for an assumed heat input. Most bridge engineers would, sensibly, be cautious of the results of such analysis because
there is a shortage of real world data to check the analysis results against, and gain confidence in the analysis method.

The Gateway Upgrade Project provided a rare opportunity to obtain some real world data on the thermal movements of a bridge of large concrete cross section.

This paper presents the results of twelve months of data on the thermal movements of the Existing Gateway Bridge, in the expectation that these results may benefit the broader bridge design community, by increasing the database of measurements against which analytical methods can be compared.

2.0 Research into the thermal movements of bridges

In the published literature on the thermal movements of bridges, the work of two researchers stands out. The first of these is New Zealand’s Nigel Priestley. During the 1970’s Priestley and his co-workers published theoretical methods for calculating thermal effects, temperature distributions to be used for design, and measurements on model bridges and actual structures (Priestley 1972, Priestley 1978, Priestley & Buckle 1979).

In the late 1970’s Mary Emerson and her colleagues at the Transport & Road Research Laboratory published the results of their analysis and measurement of bridge thermal movements in the United Kingdom (see for example TRRL 1978, Emerson 1979).

Many others have contributed to our knowledge in this area. In the USA Roeder & Moorty have been active in this field (Moorty & Roeder 1992, Roeder 2003), and returning closer to home Churchward (1981) and Hirst (1982) have published Australian research on thermal movements.

It is from the work of these researchers, and others, that our current codified methods for designing for thermal movements have developed. However, if we are to progress our knowledge further in this area, the most pressing need is not for additional or enhanced methods of thermal analysis, but for real data against which the current methods for heat flow analysis can be tested, so that we can gain confidence in their application.

3.0 Gateway Bridge geometry and instrumentation

The Existing Gateway Bridge (EGB) was constructed in the mid-1980s and is a concrete box girder bridge. The bridge is 1627 metres long between fixed abutments and has two “needle-beam” halving joints with modular expansion joints at deck level. The southern expansion joint is 376 metres from the southern abutment, and the northern expansion joint is 731 metres from the northern expansion joint. The main span is 260 metres between river piers.

The instrumentation installed within the EGB comprised string potentiometers (string pots), and resistance temperature detectors (RTD’s). These devices were connected to a Datataker DT 80 data acquisition unit. Identical instrumentation was installed at the northern and southern expansion joints. At each joint two 500mm range string pots were set between the opposing concrete faces (Figure 1) in order to measure the movement across the joint. One string pot was set towards the eastern end of the joint, and one was set towards the western end of the joint. The four string pots were labelled as follows: NE
(Northern expansion joint, Eastern side of box), NW (Northern expansion joint, Western side of box), and SE and SW respectively.

An RTD was fixed to the inside face of the end diaphragm of the box girder, in order to measure air temperature inside the box. Figure 2 shows the data acquisition unit mounted on the box girder diaphragm. Data was stored locally and downloaded periodically from the data logger to a USB memory stick.

Both movement and temperature were logged every 30 minutes. The shade air temperature at Brisbane Airport, also at thirty minute intervals, was acquired from the Bureau of Meteorology.
4.0 Summary of the data obtained

4.1 Bridge movement

At the time of writing this paper, the string pots installed inside the bridge had measured the opening and closing movements of the modular expansion joints every thirty minutes for a period of almost 14 months.

![Graph showing joint movements at northern expansion joint](image)

*Figure 3 Joint movements at northern expansion joint*

Figure 3 displays the joint displacements for the NE and NW string pots. A positive value of joint movement implies that the bridge has expanded (the joint has closed) since time zero. A negative value of joint movement implies that the bridge has contracted (the joint has opened) since time zero. The data is manipulated in such a way as to ensure that a positive gradient of displacement, with respect to time, is associated with a positive gradient of temperature with respect to time.

As the string pots (NE and NW) are adjacent and the predominant thermal movement of the bridge is in the longitudinal (north-south) direction, one would expect the movements to be almost identical. Any discrepancy in the NE and NW displacements could be attributed to in-plan bending of the superstructure caused by differential temperature effects on one side.
A linear regression of NE and NW displacements has been undertaken for over one year of data (Figure 4) and indicates a strong ($R^2 = 0.99$) correlation between the two. The correlation at the southern expansion joint, between SE and SW, is not as strong ($R^2 = 0.95$). This may indicate that differential temperature effects may be active causing in-plan bending of the superstructure, or that the data at this joint is less reliable. Further discussion in this paper, for simplicity, will use only that data obtained from the NE string pot.

4.2 Movement and shade air temperature

![Figure 5 Joint movement and temperature](image_url)
Figure 5 shows joint displacement and temperature plotted against time. An arbitrarily selected five-day period (4 December 2007 to 9 December 2007) is shown for clarity.

On a daily basis the temperature increases from a low point in the early hours of the morning, reaches a peak temperature in the early hours of the afternoon, and then begins to fall, once again reaching a trough in the early hours of the following morning.

This diurnal temperature variation is accompanied by a similar pattern in the joint displacement. As the air temperature increases through the morning, the bridge temperature also increases and the bridge expands longitudinally, closing the expansion joint. The converse is true as the temperature decreases. There appears to be a definite phase shift or “time lag”, however, which in Figure 5 is approximately 6 hours.

5.0 Comments and observations on the data

There is a phase shift between temperature and joint displacement of about 6 hours. That is, the movement response of the bridge lags behind the temperature change by approximately 6 hours, due to the thermal inertia of the massive concrete structure.

Expansion or contraction of the structure occurs as a result of changes in the bridge temperature, not the air temperature. Bridge temperature increases as a result of incident radiation (sunlight) and convection (moving air passing the concrete surface and either transferring heat to the concrete or transporting heat away).

The webs and flanges of concrete box girders are thermally massive; a substantial increase in the temperature of these elements requires lengthy exposure to either radiation and/or convection. Conversely, a structure such as a small steel girder pedestrian bridge with a light timber deck will increase in temperature far quicker under the same conditions. It will respond faster to ambient temperature changes and sunlight radiation; it has less thermal inertia.

5.1 Measured versus calculated bridge movements

The issue of thermal inertia and the presence of a clearly identifiable response time lag suggests that the bridge does not respond to the full range of temperature extremes. For example, during a hot day in summer, the shade air temperature may climb to 30 degrees Celsius; however, this temperature may only be sustained for an hour or two before the afternoon cooling begins. As this “peak” is sustained for such a short period of time, the thermal inertia of the bridge will prevent the average bridge temperature from reaching 30 degrees. Accordingly, the bridge will not fully respond to the maximum temperature of 30 degrees. Therefore, the daily temperature extremes are not realised in the bridge’s response.

To test this hypothesis, a bridge movement was calculated from the observed shade air temperatures. This was then compared with the measured bridge movements.
The calculated bridge movement is given by:

\[ \Delta_t - \Delta_0 = (T_t - T_0) \alpha L \]

where,

\( \Delta_t \) = the joint displacement at time \( t \) (mm)
\( \Delta_0 \) = the joint displacement at time zero
\( T_t \) = the temperature at any time (degrees Celsius)
\( T_0 \) = the temperature at time zero
\( \alpha \) = the coefficient of thermal expansion

\( \alpha = 1.1 \times 10^{-5} \) /degree Celsius (AS5100.5 Clause 6.1.6)

\( L \) = the length of bridge subject to expansion or contraction

The length \( L \) is taken as the distance from the northern abutment to the northern expansion joint (731 metres) plus half the distance between the expansion joints (520 metres). As such, \( L \) is equal to 991 metres.

The joint displacement at time zero (\( \Delta_0 \)) was set to the difference between the average measured and average calculated movements over the period of measurement.

Figure 6 shows an arbitrary period of almost one. The calculated movement is shown for both shade air temperature (Brisbane Airport) and the ambient air temperature inside the bridge. The movements calculated using shade air temperature generally exceed the measured movements, whereas those calculated from the air temperature inside the bridge significantly underestimate the movements. The air temperature inside the bridge appears to be of little value and is not used in subsequent discussions. A linear regression (Figure 7) further confirms the lack of fit. The former is a direct consequence of the thermal inertia of the massive concrete structure, whereas the latter reflects the fact that air temperature within the bridge is very stable and does not reflect fluctuations outside.

To allow for the observed time lag (due to thermal inertia of the bridge), the joint movements were re-calculated based upon an average rather than an instantaneous shade air temperature. Using simple linear regression, it was found that an 8-hour averaging period gave the best fit to the measured movements.
The movements calculated using 8-hour averaged shade air temperatures still exceeded the measured movements. A scaling factor was introduced to improve the correlation between calculated and measured movements.

\[ y = 0.83x \]

\[ R^2 = 0.59 \]

**Figure 6 Measured and calculated movements**

**Figure 7 Linear regression: measured versus calculated joint movements**
The calculated bridge movement is then given by:

\[ \Delta_t - \Delta_0 = K(\bar{T}_t - T_0)\alpha L \]

where,

- \( \bar{T}_t \) = moving average of shade air temperature over an 8-hour period
- \( K \) = a dimensionless scaling factor
- \( = 0.985 \)

Both Figures 8 and 9 show a stronger correlation between measured and hypothetical bridge movements and therefore indicate that a moving average air temperature is a more appropriate means of predicting average bridge temperatures and therefore, bridge movements. The averaging process "smoothes out" the extreme daily temperatures and acts as a surrogate means of dealing with thermal inertia.

For the EGB, an 8-hour moving average of shade air temperature and a scaling factor of 0.985 was found to adequately compensate for the thermal inertia effect. However, it must be stated that this figure will not hold for all structures – it merely applies for the EGB. In general, the more thermally massive a structure the greater the period required for time averaging.

![Figure 8 Measured and calculated movements for 8-hour moving average temperature](image-url)
5.2 Extreme values of bridge movement and comparison with AS5100

The current Australian Code (AS5100 – Bridge Design) contains simple rules-of-thumb for converting shade air temperatures in different regions of the country into average bridge temperatures for calculation of expansion joint movements.

If AS5100 were used to derive the temperature range for the EGB, the Region of interest is Region II (coastal) as the bridge lies south of 22.5 degrees S. As such, the maximum shade air temperature is 44 degrees Celsius, and the minimum shade air temperature is -1 degrees Celsius.

If no allowance is made for the heat sink effect, AS5100 would require a maximum bridge temperature of 48 degrees Celsius and a minimum bridge temperature of 4.7 degrees Celsius. The Code-predicted temperature range is, therefore, 43.3 degrees Celsius.

Over the period of 14 months of data from the dataloggers, the maximum measured joint movement was 124.9 mm and the minimum joint movement was – 128.1 mm; a movement range of 253 mm. Accordingly, the apparent average bridge temperature range (that is, the temperature range calculated from the measured movements) for this period was

$$\Delta T_{\text{bridge}} = \frac{253}{1.1 \times 10^{-5} \times 991 \times 10^3} = 23.2^\circ C$$

The maximum shade air temperature during this period was 39 degrees Celsius and the minimum shade air temperature was 3 degrees Celsius – a temperature range of 33 degrees Celsius. Table 1 summarises this information.
With respect to bridges such as the EGB, with high thermal inertia, these AS5100 rules appear to over-predict the average bridge temperature range for a given shade air temperature range, unless the heat sink effect is considered. Currently, AS5100 provides the only commonly accepted means of calculating average bridge temperature ranges for Australian-designed bridges. These results highlight the scope for further work and investigation to provide more accurate means of estimating this important design parameter.

<table>
<thead>
<tr>
<th>Table 1 Shade air temperature ranges and average bridge temperature ranges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shade air temperature range (deg Celsius)</td>
</tr>
<tr>
<td>------------------------------------------</td>
</tr>
<tr>
<td>33</td>
</tr>
<tr>
<td>Average bridge temperature range (deg Celsius)</td>
</tr>
</tbody>
</table>

6.0 Conclusions

The authors make the following conclusions regarding the thermal movement of the EGB:

- Expansion joint displacements lag behind changes in air temperature by about 6 hours;
- Prediction of expansion joint displacements using the daily peak and trough air temperatures grossly overestimates the movement of expansion joints;
- Thermal inertia is responsible for the time lag and the fact that movement ranges do not reflect extreme air temperature ranges;
- A reasonably accurate prediction of expansion joint movements using air temperature is achieved by taking an 8-hour moving average of air temperature to “smooth out” the peaks and troughs and scaling the calculated movements by 0.985;
- The Australian Standard AS5100 is conservative in the prediction of average bridge temperature ranges for large bridges such as the EGB unless the heat sink effect is considered. There is scope for further work to be done to produce a more accurate means of predicting average bridge temperature ranges for large bridges.

7.0 Acknowledgment

The authors wish to thank Queensland Motorways Limited, Leighton Abigroup Joint Venture, and the Maunsell-SMEC Joint Venture for permission to publish this paper. The views expressed in this paper are those of the authors.

8.0 References


Emerson 1979 “Bridge temperatures for setting bearings and expansion joints” TRRL Supplementary Report 479


Priestley & Buckle 1979 “Ambient thermal response of concrete bridges” Road Research Unit, National Roads Board, New Zealand

Priestley 1972 “Thermal gradients in bridges - some design considerations” New Zealand Engineering Vol 27 No 7 July 1972

Standards Australia 2004 “AS5100 Bridge Design”

TRRL 1978 “Bridge temperatures” TRRL Supplementary Report 442