RAILWAY & HARBOUR DIVISION

THE OLIFANTS RIVER BRIDGE - THE INHERENT PROBLEM

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SUMMARY

The Olifants River Bridge causes many difficulties to track personnel if not managed sufficiently. These issues are caused due to the length of the structure, as well as the location of its expansion joints combined with the use of CWR (Continuous Welded Rail). This paper will discuss the history and events that occurred at the bridge, the issues causing high concentration of rail forces, measurements employed in order to manage the rail stresses on the bridge as well as the technologies that will be considered in the future.

The history will include events leading up to a derailment and the actions taken thereafter, leading up to the installation of a measurement system. The measured forces, temperatures and deck movement will be discussed in terms of the data gathered over time. The current practices on the Olifants River Bridge and various other possibilities will also be discussed.

The paper concludes with a summary of the most important findings from the analysed data as well as specific indications of future methods and technologies that will be introduced on the bridge.

1 INTRODUCTION

The Olifants River Bridge is situated on the Iron Ore Export Line between Sishen (Northern Cape) and Saldanha (Western Cape). This is a 30 tonne per axle single line which is currently transporting in the order of 60 million tonnes of iron ore annually. The bridge is located where the railway line moves away from the coastal area and passes over the Olifants River near the town of Vredendal. This town is notorious for the extreme ambient temperatures reached there.

The bridge itself has a length of 1035 m and has a height of 51 m at the highest position. It consists of three concrete decks, the two at the abutments with lengths of 495 m and the middle deck with a length of 45 m. It has 23 spans of 45 m each as can be seen in Figure 1. The 495 m deck segments between the fixed supports and the expansion gaps are allowed to move on bearings for thermal expansion and contraction.

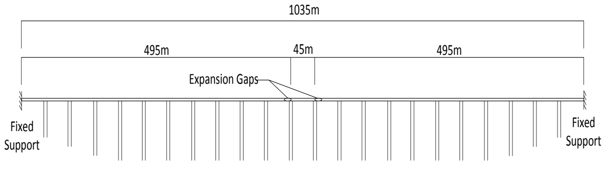

Figure 1. Olifant River Bridge layout

Due to the use of CWR (Continuous Welded Rail) on the bridge combined with the forces induced by deck movement, significant force concentrations are generated in the rail at the positions of the expansion gaps.

2 NOTATION

The following symbols are defined as used in the rest of the paper:

P = Force in the rail due to thermal change

A = Cross-sectional area of the rail

α = Coefficient of thermal expansion

SFT = Stress Free Temperature of the rail

RT = Rail Temperature

F = Friction force between ballast and bridge deck

N = Normal force caused by weight of track structure

μ = Coefficient of friction

w_{avg} = Average frictional force per millimetre of deck movement

F_i = Bridge deck force at timestamp i

δ_i = Bridge deck movement at timestamp i

3 HISTORY

3.1 Design and construction

The bridge construction was started in July 1973 as part of the initial construction of the Iron Ore

Export Line [1]. This was completed in 20 months' time [2]. The line was built in 1974 by ISCOR (Iron & Steel Corporation) and later taken over in 1977 by SAR&H (South African Railways & Harbours) which is now Transnet Freight Rail [3].

The bridge was designed by Spie Batignolles, consultants from France. The construction method used was an all-time first in South Africa. The segments were incrementally constructed and after each constructed span, it was sequentially launched. The entire bridge was built and launched from the western abutment [1].

In Europe, when designing continuous deck structures, the issues with regards to deck movement and the forces it induces are usually prevented by limiting the permissible length of such a structure [4]. Although this design was well beyond the limiting length of 200 m, it was accepted at that time as a construction period of only 17 months were offered [2].

Due to the length of the continuous decks, the bridge deck movement is quite significant. The interaction between the bridge deck and the ballast causes additional forces to be induced in the rail. The friction between the ballasted track and the concrete deck that causes the additional forces were approximated as 9 kN per millimetre movement of the deck per metre of track based on European standards. The resistance value later established by the Bridge Office was 14 kN/mm/m [4].

3.2 Derailment

On the 24th of September 1982 a derailment occurred on the bridge due to the lateral buckling of the track. This could be attributed to a number of factors.

The infrastructure maintenance depot at Saldanha had an opportunity to do some on-track maintenance due to a delay in the train service caused by an issue in the port. The decision was made to do some tamping on the bridge. This work was done on the 16th of September, a week prior to the derailment. In 1982 the line was still carrying 26t/axle and much shorter trains of 70 wagons compared to the current 342 wagons per train. Thus it was approximated that the track was only 38% consolidated at the time of the derailment [5]. This contributed to the de-stabilisation of the track by reducing the effect of shoulder and crib ballast. The reduced friction area between the contributing ballast zones and the sleepers decreased the lateral resistance of the track.

A simple analysis of vertical reactions under typical ore train loading of 1982 indicated a reduction in the downward normal force on the sleepers under the wagons (see Figure 2). This also means that the friction between the ballast and the sleeper is reduced. Various sources suggest that the top ballast contributes 30% to 60% of the lateral resistance of the track superstructure [4, 6, 7]. Thus the kick out most probably occurred under a wagon of the loaded train.

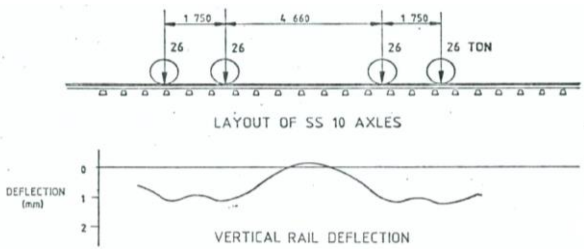

Figure 2. Reduction in downward normal force (taken from [4])

The final contributing factor was the temperature induced forces – being caused by the rail and the deck movement. The mean air temperature rose by approximately 10°C over a period of ten days, which is 60% of the total seasonal temperature change [3, 4]. The maximum air temperature was measured at 42°C causing a rail temperature of approximately 61°C. With the SFT (Stress Free Temperature) of the rails on the bridge set at 30°C, it would cause a longitudinal compressive rail force in the region of 550 kN. This gives a track force of 1100 kN.

Various measurements were taken on the Olifants River Bridge between 1979 and 1981 by the Bridge Office and the then Track Development Office respectively. The air and rail temperatures as well as deck movement were extensively measured since 1979. Figure 3 shows the temperatures at the time of derailment, overlayed on the measurements conducted by the Bridge Office. The Track Development Office used vibrating wire strain gauges to measure the strains and the corresponding rail forces before the current monitoring system was installed in October 1982.

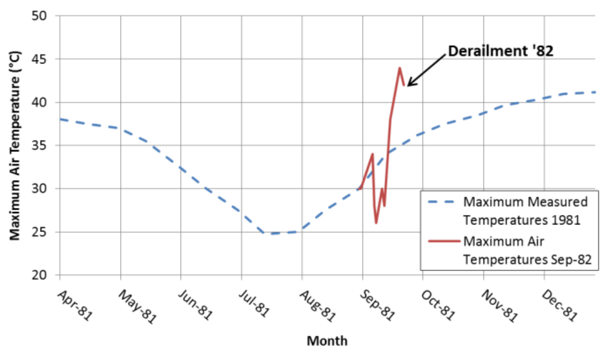

Figure 3. Maximum air temperatures (adapted from [4])

After the occurrence of the derailment, the measured data was used to approximate the expected track forces at the time of the derailment. This was found to have reached a peak value of about 3000 kN at the time of derailment, which is the combination of CWR (1100 kN) and deck forces (1900 kN) with the deck force accounting for about 60% of the total track force [4].

3.3 Actions implemented

3.3.1 Track changes

Various methods were considered to prevent another event as the one described in Section 3.2. Some were immediately implemented while others first had to be implemented on the open track where it holds fewer risks.

PY winged sleepers were installed over the entire length of the bridge in order to increase the lateral resistance of the track. It was mentioned that these sleepers can increase lateral resistance by 100% [8]. Apart from the change in sleepers, the track was lowered in order for the sleepers to be below the height of the ballast retaining walls (see Figure 4). The now reduced ballast depth under the sleeper will not be ideal in terms of distributing the vertical load to the structure.

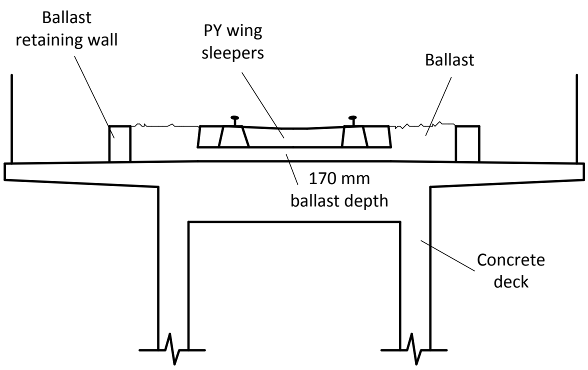

Figure 4. Olifants river bridge cross-section

Different fastenings were installed as an experimental section on the open track. The fastenings were to allow for longitudinal movement while still maintaining the proper gauge and restraining it in the lateral and vertical directions. These fastenings were accompanied with the use of a splice joint to allow for the movement due to expansion or contraction. The clips were similar to the normal e-clips, but were installed to have a one millimetre gap between the clip and the foot of the rail, while still keeping the gauge clips in position. A rail force was produced in the rail by letting a locomotive perform an emergency brake application on the down gradient. The rail movement at the splice joint was insufficient and the fasteners did not maintain the required 1 mm gap. The splice joint was considered a risk for installation on the bridge, due to the required maintenance and significant longitudinal movement it should allow for.

3.3.2 Measurement system

A measurement system to continuously monitor the rail forces was required. This type of system was already planned before the derailment occurred. It was only installed in October of 1982 [5].

This measuring system had four strain gauges on each rail at both expansion joints in order to measure the rail forces. A total of sixteen strain gauges were located on the rail in order to provide sufficient redundancy. These gauges were then calibrated by de-stressing the track and hereby zeroing the readings. Apart from measuring the rail forces, various other parameters were also measured. The latter included air temperatures, rail temperatures, concrete deck temperatures and longitudinal deck movement.

Certain alarm conditions were also introduced in order to prevent damage to the bridge, track structure or unrecoverable disruption in the train service. These alarm limits had different levels each with their respective actions to be taken. The first alarm limits, which are 800 kN for compression forces and 1300 kN for tensile forces, imposes a speed restriction of 15 km/h over the bridge, which also requires the track to be inspected before a train is allowed to proceed. Once the limits of 900 kN for compression and 1500 kN for tensile forces are reached, the trains are not allowed to traverse the bridge until forces have dropped below these limits [9].

4 MEASURED DATA ANALYSIS

4.1 Monitored temperatures

The measured temperatures provide important information to improve the understanding of the bridge’s performance. It shows how the increased air temperatures are followed by a rapidly increasing rail temperature as well as a delayed peak temperature for the concrete deck. The latter also causing a delayed deflection (see Figure 5) and the corresponding increase in rail force. The rate at which heat are transferred to the concrete is also influenced by factors like the wind, especially when it is blowing from the south-west (from the coast).

One of the most important elements that can be noticed by analysing the data is the effect of consecutive warm or cold days. As mentioned, the concrete deck has a delayed temperature peak and expansion, which is due to the low thermal conductivity of concrete. Concrete's thermal conductivity is between 0.4 and 1.8 W/(m K) compared to steel's value of 43 W/(m K) [10]. This property is a double edged sword. In the case of a single day of extreme weather the deck would not expand enough to induce excessive rail forces, but consecutive extremes cause deck movement that cannot recover in a day's time. The effect of consecutive days can be seen in Figure 6.

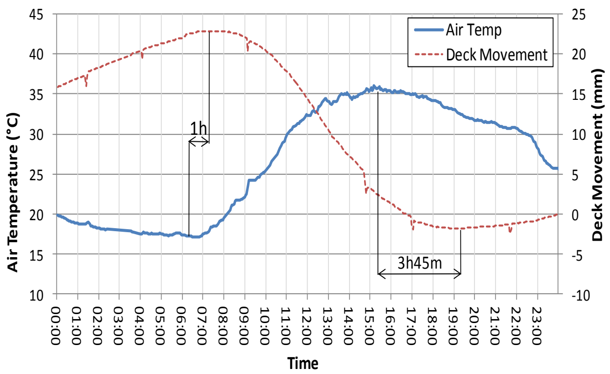

Figure 5. Delay in deck movement

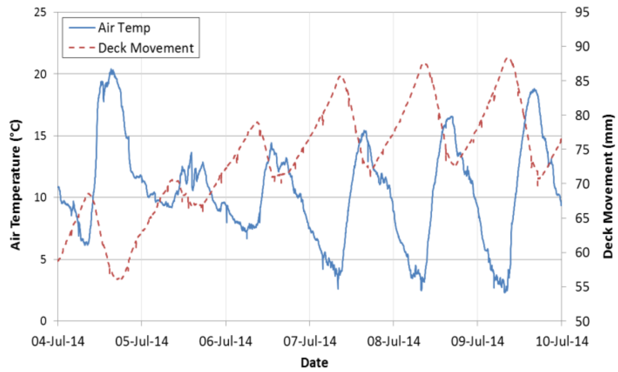

Figure 6. Influence of consecutive extremes on concrete deck movement

4.2 Continuous welded rail forces

In order to illustrate the major difference between the rail forces on the Olifants River Bridge compared to open track, the effect of CWR will be briefly explained. The CWR forces are thermally induced whenever the rail temperature is above or below the SFT of the rail, where the SFT is defined as the rail temperature at which rail is fastened stress-free to the sleepers. The CWR forces can be calculated by means of the following equation:

$$P = A.E.\alpha.(SFT - RT) \;\; [Eq.1]$$

The CWR forces for the rails on the bridge were calculated with an SFT of 32°C to which it was previously de-stressed. In Figure 7 it shows the CWR forces in a dotted line compared to the total rail force on the bridge. This clearly shows the forces that would be experienced on the open track compared to the combination of forces on the bridge.

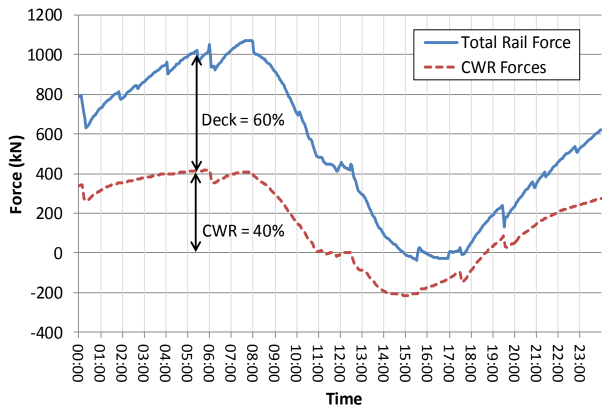

Figure 7. CWR and deck contribution to total force

4.3 Bridge deck forces

The expansion of the two long decks (495 m) which are fixed at the abutments cause severe forces in the rail at the positions of the bridge expansion gaps. This is also the reason why the measuring system is installed at those specific positions. The force due to the deck movement is fundamentally transferred to the rail via the friction between the ballast and the concrete deck as mentioned in Section 3.1. The basic equation for friction force can be used to clarify this:

$$F = N.\mu \;\; [Eq.2]$$

This is a function of the weight of the track which will determine the normal force acting on the deck, while the condition of the ballast will most likely contribute to the value of the friction coefficient. The total movement of these expansion gaps in a year is up to 118 mm as recorded in 2014. Figure 8 shows the force induced by the bridge deck movement. This has been calculated by subtracting CWR forces from the measured rail forces. The force spikes also evident on the graphs are caused by train movement over the bridge. The actual forces caused by the bridge deck were approximated to be 24 kN per millimetre of deck movement as will also be shown in Section 5.1.

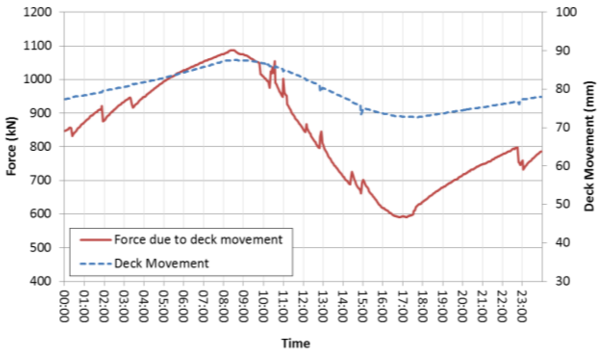

Figure 8. Force due to deck movement

Due to the sheer length of the deck and therefore the significant change in length, concentrated loads are generated in the rail at the bridge expansion gaps. An approximation of this is shown in Figure 9.

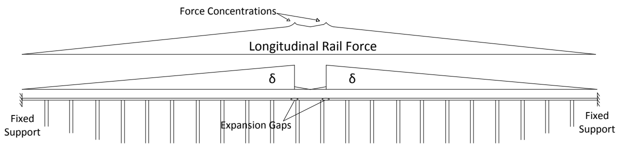

Figure 9. Rail force concentrations

Apart from the concentrated forces caused by the deck movement, this mechanism also causes deterioration of certain components of the track superstructure. The ballast in the vicinity of the expansion gaps break down faster than in other areas, due to the additional abrasive action caused by the daily and seasonal cyclic longitudinal movement. This leads to an increased rate of ballast fouling as shown in Figure 10.

Figure 10. Ballast fouling at expansion gaps

The breakdown of ballast also causes slacks to form at the expansion gaps, which requires it to be lifted out. The abrasive action between ballast and sleeper also causes the deterioration of the sleepers. The presence of the force concentrations is also evident in the movement of sleepers as can be seen in Figure 11, where the dotted red lines represent the location of the expansion gaps. These measurements were recorded in March 2013.

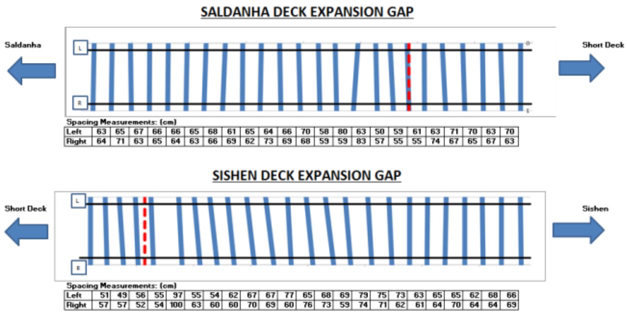

Figure 11. Sleeper movement due to forces

5 MAINTENANCE INTERVENTIONS

5.1 Sleepers and ballast

Various actions have been taken in the last few years to maintain the track on the bridge. These ranged from basic tasks like lifting of slacks as mentioned in Section 4.3 to major actions taken during the Iron Ore Export Line's annual shutdown in August of 2013.

During the shutdown a total of 50 winged PY sleepers and 80 m of ballast were replaced in the locations of the expansion gaps. This was required due to the deterioration of the track components. It was also important to ensure that the tamping of the track was done properly and correct the first time. Track geometry had to be within the A-standard and the sleepers were required to be within the height of the ballast retaining walls for additional lateral resistance as mentioned in Section 3.3.1.

The replacement of ballast was expected to reduce the resistance between the concrete deck and ballast and thus the force induced in the rail per millimetre of deck movement. Analysis of three months before and after actions taken in 2013 showed no notable change in this regard. Table 1 below shows the averages calculated per month based on the calculated daily force per deck movement. This was calculated with a simple gradient equation:

$$w_{avg} = \frac{\Delta F}{\Delta \delta} = \frac{F_B - F_A}{\delta_B - \delta_A} \quad [Eq. 3]$$

Table 1. Average resistance force (Year 2013)

Months Before Shut	Average Resistance [kN/mm]	Months After Shut	Average Resistance [kN/mm]
Jun	23.9	Sep	24.3
Jul	23.3	Oct	24.3
Aug	22.9	Nov	23.4

The expected results in terms of resistance per millimetre were probably not achieved due to the short distance of ballast that was replaced (see Figure 12). Only 17.5 m of ballast in the direction of the long decks were replaced, where the force caused by the bridge deck builds up over almost the entire 495 m. In the coming shutdown of 2015 the entire bridge is scheduled to be screened, which will most certainly produce interesting results.

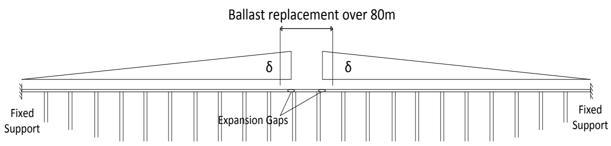

Figure 12. Ballast replacement

5.2 Rails

In order for the alarm limits as mentioned in Section 3.3.2 to be valid, the rails on the bridge should be replaced between 500 and 600 million gross tonnes [9]. This amounts to rail replacement every 8 years at the current tonnages. This will need to include the reinstallation of the measuring system's gauges.

The de-stressing of the rails on the bridge can only be done in limited periods, where the expansion gaps are close to its neutral positions and the rail temperature is within the A-range. A tensor can also be used, but the period is still limited due to the force induced by deck movement. The de-stressing is usually only done when the measuring system requires a calibration. In other cases a method of force re-distribution is used to relieve the stress on the Olifants River Bridge.

5.3 Force re-distribution

Generally the distribution or re-distribution of track forces relates to the removal of fastenings over a limited length to allow for rail movement. This is an action taken in order to alleviate rail stress concentrations. The idea is to execute this action as the mean temperature gradient shifts sufficiently, e.g. at the onset of summer and winter. This action may however in the case of un-seasonal temperature changes intensify the problem [4, 8].

Before taking this action it is important that the forces in the rail are monitored and the planning is done properly. The purpose would be to effectively lower the SFT over the section with loosened fastenings for the winter period and effectively increase the SFT over the same section for the summer.

This action was recently performed for the summer of 2015 (Dec - Feb). The air temperature during this summer was relatively normal, although in the month of March consecutive days of extreme temperatures were recorded. Air temperatures were in the order of 42°C and significant bridge deck movement did cause the rail force to reach -800 kN. Usually this type of temperatures will cause the closure of the bridge, but the re-distribution seemed to have prevented it in this case.

6 CONCLUSIONS

The objective of this paper was to stress the problem caused by the Olifants River Bridge due to the way it was designed. It also discusses the actions taken, as well as the systems and procedures put in place to prevent derailments.

The causes of the stress concentrations in the rail at the bridge expansion gaps include the following:

- Continuous decks of 495 m that have maximum expansion and contraction at the expansion gaps.
- CWR forces contributing to the maximum rail force.
- Extreme temperatures which occur for consecutive days causing maximum rail forces and concrete deck movement.

The preventative actions taken, as well as the systems and procedures that was put in place include the following:

- PY winged sleepers were installed which increase lateral resistance.
- The track superstructure was lowered in order for side walls to provide lateral resistance.
- A measurement system, providing detailed data in order to monitor various parameters, was installed.

- A method of re-distributing the forces in the middle of the bridge was implemented.

7 RECOMMENDATIONS

Based on the preceding conclusions, the following recommendations were made:

- Limit the continuous length of bridge decks in order to limit the problems in the future.
- In the case of having lengthy decks, ensure that there is a monitoring system in place as it greatly assists in managing the problem.
- Inspecting, monitoring and maintaining track assets on a regular cyclic basis are of utmost importance.

8 ACKNOWLEDGEMENTS

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