Forces in the railway track structure on the Olifants River Bridge

by Hannes Maree

The Olifants River Bridge is unique in that it carries continuously welded rails over its entire length of 1035 m (3396 ft). The thermal expansion of the decks and the rails interact and therefore superimpose forces on to the rails. Due to the extreme variation in the daily temperature, these forces will build until a relative movement is produced between the rails and the bridge deck. Should these forces become high enough the rails buckle and cause a derailment of a passing train. A special monitoring system consisting of strain gages, carrier frequency amplifiers, and a scanning system was installed to prevent further derailments.

The Olifants River Bridge, shown in **Fig. 1** is on the Sishen-Saldanha (Rep. of South Africa) rail line. This line is dedicated for the transport of iron ore. The Olifants River Bridge is unique in that it carries continuously welded rails over its entire length of 1035 m (3396 ft). Fig. 1 also shows a simplified diagram of the bridge. The structure consists of two 11 x 45 m (11 x 148 ft) continuous concrete side spans fastened at the abutments and a simply supported 45 m (148 ft) concrete span in the middle. The long decks are free to expand or contract on either side of the center deck by means of expansion joints. The rails are therefore subject to high interaction forces. Concern over the magnitude of these forces resulted in the development of a system for their measurement.

Direct measurement of the strain is difficult due to the high values of stress. Various types of strain measuring devices were tried such as vibrating wire gages, spot-welded strain gages and HBM type DA 2 glued on strain gages. It was found that the DA 2 glued on strain gages provided satisfactory results. European Railroad Authorities solve the measurement problem by limiting the length of continuous deck structures. The maximum permitted lengths vary widely between authorities. The longest permitted length is 200 m (656 ft) for ballasted track with continuously welded rails.

Ort September 24, 1982 shortly before the installation of the measurement system, a derailment took place in the center of the bridge. **Fig. 2** shows a bottom view of the bridge and one of the damaged rail cars at the bottom. Approximately one million US dollars were lost due to the derailment. The cause was a kickout of the track due to the excessive compressive forces in the rails. In October 1982, the measurement system was finally put into service.

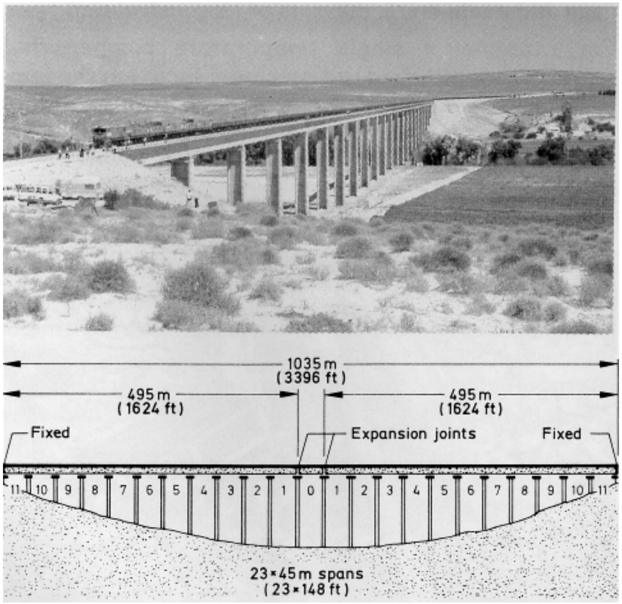

Fig. 1: Olifants River Bridge between Sishen and Saldanha, in South Africa. Photograph and simplified diagram of the bridge

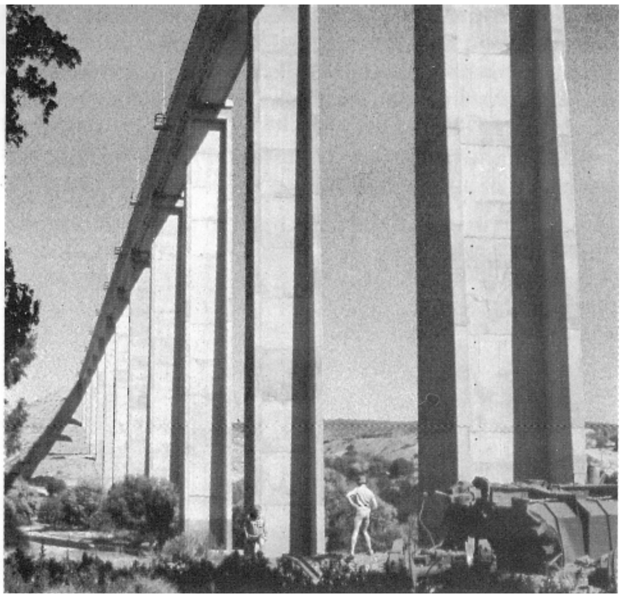

Fig. 2: View of the Sishen-Saldanha Bridge from below showing one of the damaged rail cars from the derailment on September 24, 1982

Interaction forces

Since the rails are continuous over a relatively long distance, thermal movement is hindered in the longitudinal direction. Depending on the temperature this produces tensile or compressive forces in the rails. In the areas near the bridge expansion joints the thermal movement of the deck interacts with the continously welded rails and results in additional forces which are superimposed on the rail forces. Due to the massive concrete decks the forces in the rails can increase until a relative movement is produced between the rails and the bridge deck. The rails are therefore subject to a buildup of interaction forces.

Method of Measurement

A copper plate calibrated at 5 mm (197 mils) intervals and glued to the rails was used to measure the relative movement between the rails and the bridge deck. A length of piano wire was stretched between the hand rails and acted as a reference point for the bridge deck. The relative movement was only measured on the Sishen side of the bridge as the hand rails were torn away from the Saldanha side during the September 1982 derailment. The movement of the long deck was monitored at the expansion joint by using a vernier calliper. The rail and air temperatures were measured with two automatic recorders.

Fig. 3 shows where the strain gages were applied to the rails and where the relative movement between rails and deck was measured. Due to their long term stability, HBM 5 kHz carrier frequency measuring amplifiers were used. **Fig**. **4** shows the set-up of the four type KWS 673.D4 CF amplifiers with a total of 24 available channels which were located within the bridge deck. **Fig. 5** shows the rail tensor from Saldanha which was used to calibrate each measurement point. The measurement points were calibrated in a stress free condition by removing the sleeper fastenings, cutting the rails and placing them on rollers. Each of the measurement points showed a linear correlation between the strain gage output and the force applied. The calibration of the rail tensor (compressor) was also checked by means of a proving ring.

In order to bring all the rails back to the same temperature base, the rails on the bridge were destressed using a rail tensor to a temperature of 38°C (100°F). The temperature range for destressing is between 25°C and 45°C (77°F and 113°F). To destress at 38°C, a stress is applied to the rails in such a manner that the actual strain in the rails is equal to the strain which would occur naturally if the rails were free to expand at the desired temperature (38°C). All the instruments were then set to zero with the rails in this position.

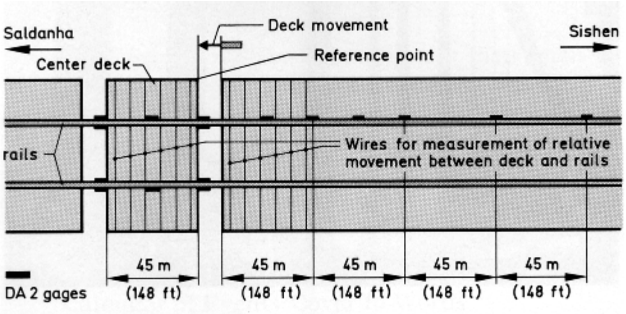

Fig. 3: Diagram showing the location of the DA 2 encapsulated strain gages and the location where the bridge deck and the rail movement was measured

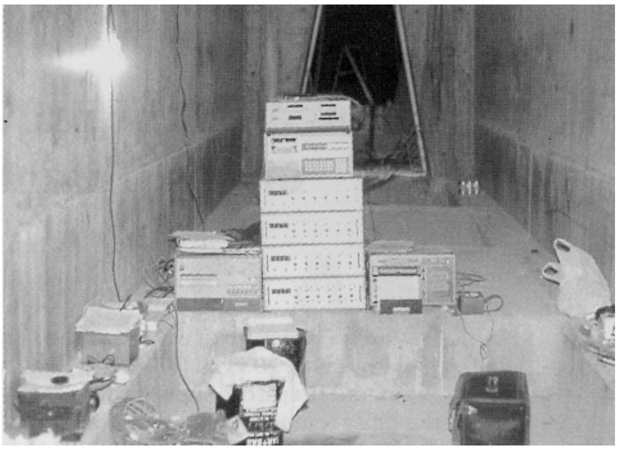

Fig. 4: Set-up of the four 6-channel CF measuring amplifiers KWS 673.D4 located within the bridge deck

DA 2 encapsulated strain gage

The DA 2 strain gage is well suited for the measurement of stresses in steel. The strain gage itself is temperature compensated for steel and is quite insensitive to temperature induced strains. The DA 2 also has a steel internal reference plate. The reference plate has two strain gages attached to it in parallel such that any bending stresses in the reference plate are eliminated while also compensating for residual temperature stresses. **Fig. 6** shows the DA 2 encapsulated strain gage and a schematic diagram of its construction.

For example: a single temperature compensated strain gage is attached to a specimen such that the temperature expansion of the specimen itself is totally free; the strain gage will then indicate approximately

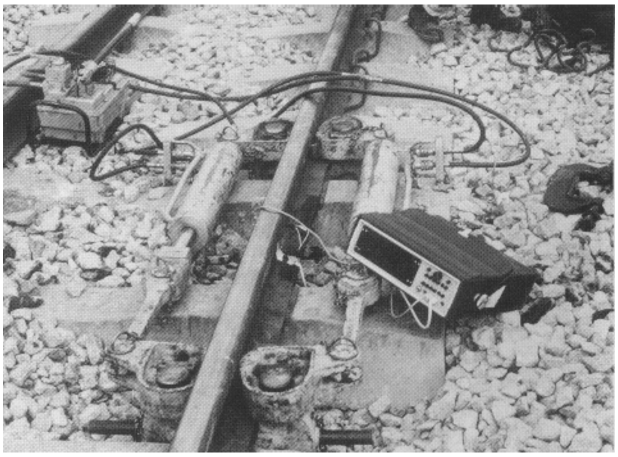

Fig. 5: Calibration of the measuring points using a rail tensor

zero apparent strain. When a second strain gage is attached to a specimen of the same material and is also allowed to freely expand, the residual temperature stresses will also be compensated. In other words the strain gage pair will produce essentially a zero output signal over a large temperature range.

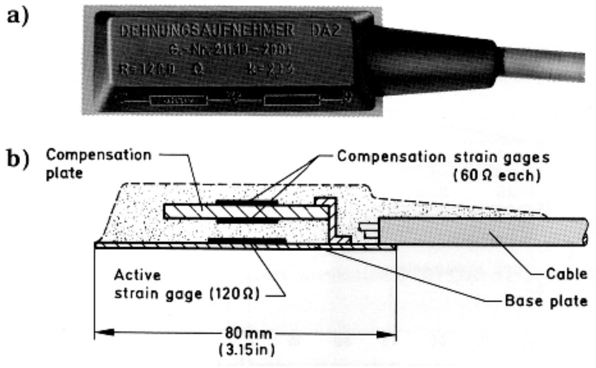

Fig. 6: The DA 2 temperature compensated encapsulated strain gage for steel
a) DA 2 encapsulated strain gage
b) Schematic cross section

If the first strain gage is restrained such that the temperature expansion is not allowed to occur or is partially hindered, the output signal measured will then be proportional to the difference between the hindered strain and the strain if the specimen were free to expand. This generally corresponds to the amount of stress applied to the specimen. This restrained strain can be accurately measured if the following restrictions are observed:

- Both strain gages are matched (k values, resistance etc.)
- The specimen has been properly cleaned
- The strain gages have been properly aligned and applied to the specimen with the proper adhesive for the desired temperature range
- The two strain gages are connected into a half bridge circuit
- The second strain gage has been attached to a specimen with the same temperature coefficient (preferably the same material) as the first
- Both strain gages are held at the same temperature
- The second strain gage is free from external stress

The DA 2 encapsulated strain gage simplifies the mounting of temperature compensated strain gages in that the previously mentioned restrictions have been followed; the user has only to properly apply the strain gage housing to a steel specimen in the desired direction.

Monitoring periods

During October and November 1982 half hourly monitoring was done by personnel from Track Development and personnel from the System Civil Engineer at Saldanha. The continuous monitoring was done to establish the friction forces between the track structure and the bridge deck. From November 1, 1982 to February 1, 1984 two survey assistants did manual recording of the bridge deck movements. They also monitored the forces in the rails as the trains crossed the bridge to establish the destressing effect or redistribution of the interaction forces due to vibrations from passing trains. They maintained radio contact with Saldanha to report forces exceeding set limits especially while repair work to the damaged decks was in progress.

Results

The structure of the bridge decks can be broken down into two distinct measurement areas: the approximately 80 m (262 ft) next to the bridge expansion joints on both sides of the center deck and the remainder of the bridge.

In the areas of the bridge decks far from the center, the temperature expansion of the rails is hindered by the massive deck structure. Since the mass of the deck is much greater than that of the rails, the strain in the rails is approximately equal to that of the deck.

Since the bridge decks are fastened at the abutments they are free to expand or contract only at the expansion joints. Therefore, this movement of the concrete decks can only occur at both sides of the center deck. However, the rails are continuously welded over the entire length of the bridge and do not have the possibility for free temperature expansion (i.e. the rails are fixed and therefore can not move).

The rails are not directly attached to the bridge deck but are fixed to concrete sleepers. The sleepers themselves are held together with crushed stone ballast. **Fig. 7** shows a view of the rails and the concrete sleepers imbedded in crushed stone ballast. This system is resting on the bridge deck.

Longitudinal deck movements from temperature changes create interaction forces. The relationship between the forces and their location with respect to the bridge is shown in **Fig. 8**. The solid line represents the track forces measured in a rising temperature cycle and the dotted line represents the forces for a falling temperature cycle.

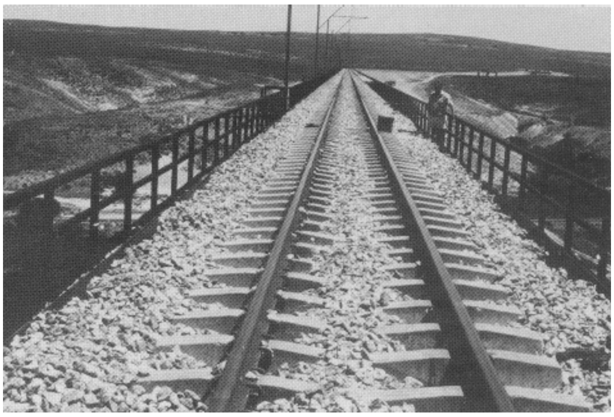

Fig. 7: View of the rails and concrete sleepers imbedded in crushed stone ballast

Fig. 9 shows the distribution of force and movement between deck and rail in the middle of the bridge for a deck movement of 16 mm (0.63 in).

As shown from **Fig. 9b** the 16 mm (0.63 in) deck movement produces an interaction force of 1200 kN (2.7 · 10^5 lbf) in two rails. It can be shown from the distribution that the friction forces between the track and the deck in a longitudinal direction is about 7 kN/m (480 lbf /ft).

Fig. 9 c shows the absolute and relative movements as measured on November 3, 1982. Since the rails can not move in the sense that they are continuously welded, the movement therefore refers to the relative movement between the deck and the rails.

The relative movement, and therefore the force is distributed along the length of the deck in the approximately 80 m (262 ft) adjacent to the expansion joints.

Since the restriction of strain causes stresses the value of the stress in the steel rails can be described by the following.

If ε_S and ε_C represent the thermal strains in steel and concrete, the value of the steel rail stress σ_S is equal to

$$\sigma_s = (\varepsilon_s - \varepsilon_c)\, E_s \tag{1}$$

where E_S is the modulus of elasticity for steel. Introducing the thermal expansion coefficients for steel α_S and concrete α_C we obtain

$$\sigma_s = (\alpha_s\, \Delta T_s - \alpha_c\, \Delta T_c)\, E_s \tag{2}$$

where ΔT_S and ΔT_C are the changes in temperature for steel and concrete respectively. Using the cross sectional area of the steel rail A_S the force F_S in the rail can be written as

$$F_s = \sigma_s\, A_s \tag{3}$$

Fig. 10a, b, and **c** show the long term effects for 1983. These graphs are all averages of maximum and minimum values for a month. There is excellent correlation between change of air temperature, deck movement and the interaction forces. This is clear from the sharp drop of temperature during April.

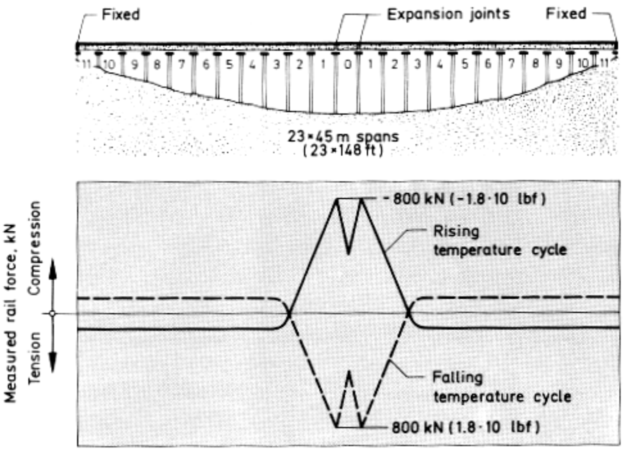

Fig. 8: Diagram showing the forces in the rails with relation to the position on the bridge deck

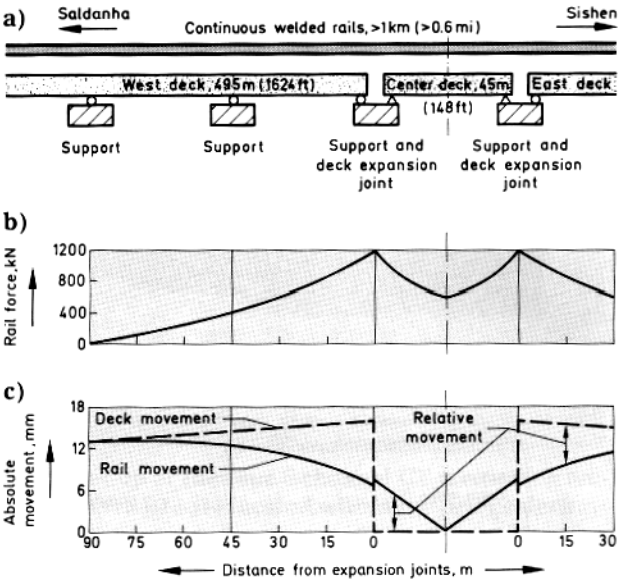

Fig. 9: Distribution of force and movement between deck and rail in the middle portion of the bridge
a) Simplified bridge structure
b) Force measured in two rails
c) Absolute and relative movement of the concrete decks and rails

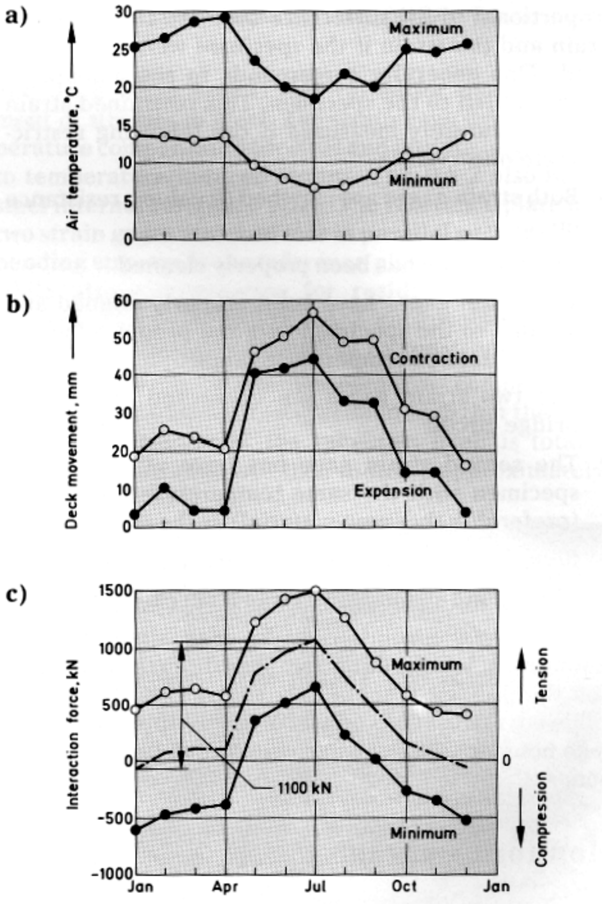

Fig. 10: Long term effects for 1983 shown as monthly averages of daily maximum and minimum values
a) air temperatures
b) deck movement (expansion joint width changes)
c) force in two rails

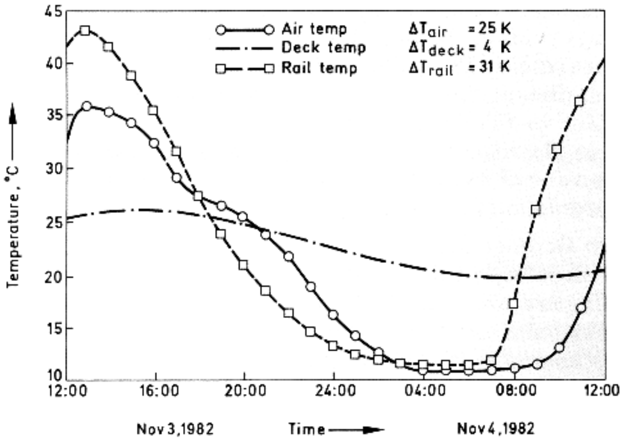

Fig. 11: Temperatures as measured on the 3rd and 4th of November 1982

From **Fig. 10b** it follows that the maximum average deck movement for 1983 was only 55 mm (2.2 in). Daily movements, not shown, indicate a maximum movement of 90 mm (3.5 in). An inspection of the bearing pads at the expansion joints clearly showed movement of 210 mm (8.3 in) in the past. When compared to other years the temperatures for 1983 are considered mild. **Fig. 10c** shows the total interaction force. Therefore, the average interaction force is the difference between the maximum and the minimum forces. The long term interaction force F_{lt} for 1983 therefore varies by about 1100 kN (2.47 · 10^5 lbf).

As an example **Fig.11** shows the results of temperature measurements recorded on the 3rd and 4th of November 1982. The forces measured in the same time period are given in **Fig. 12**. Fig. 12 also shows the effect of passing full and empty trains.

The vibrations caused by the trains lowers the track forces by spreading them out over a longer distance. Due to this redistribution of track forces the long term interaction forces do not increase proportional to the effective deck temperature. From measurements of rail forces taken before, during and after the passing of trains, it follows that the interaction forces are reduced by 40 to 50 % in the direction of "zero".

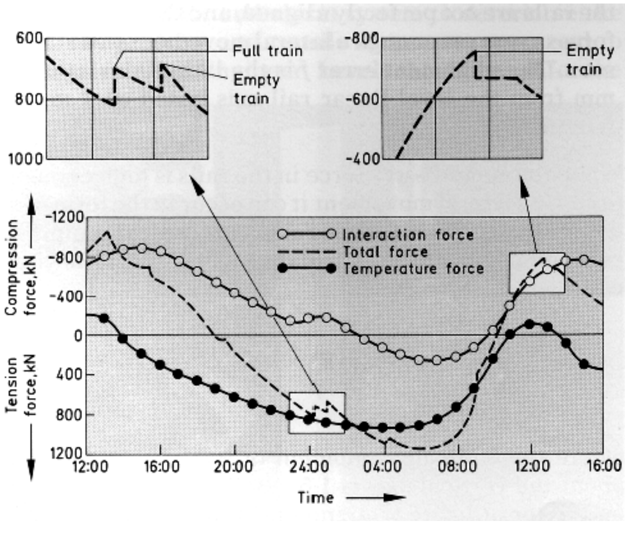

Fig. 12: Rail forces as measured on the 3rd and 4th of November 1982. The effects of passing trains on the total measured force is clearly seen

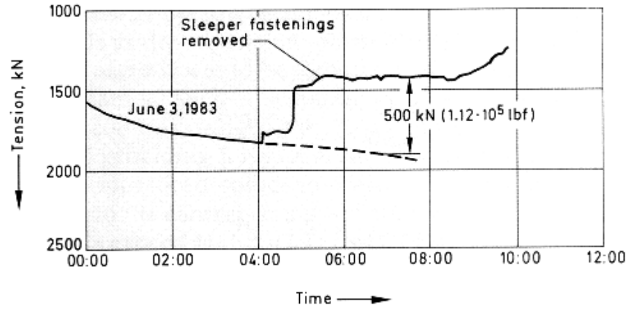

Fig. 13: Diagram showing the reduction in the rail forces when the sleeper fastenings were removed

During May 1983 the tensile forces in the rails rose to 1500 kN (3.37 · 10^5 lbf) per rail. This corresponds to a rail stress of 197 N/mm^2 (4.11 · 10^6 lbf/ft^2) and a strain of 963 ·10^{-6}. With friction forces of 7 kN/m (480 lbf/ft), a break in the rails at the expansion joints could cause a gap great enough to cause a derailment.

On June 3, 1983 the sleeper fastenings of the 400 m (1300 ft) in the center of the bridge were removed and the rails placed on rollers. The deck contracted without transfering any force to the rails. **Fig. 13** shows that tre rail force was reduced by 500 kN (1.12 · 10^5 lbf). The stress free temperature of the rails was then reduced by 10K (18R) from 38°C to 28°C (100°F to 82°F) without cutting the rails.

In **Fig. 14** the daily expansion of the deck is plotted against the rise of air temperature. Most of the points are situated in a band which has a slope between 0.95 and 1.3. These results can be expressed by:

$$\Delta l = \alpha' l \, \Delta T_{air} \tag{4}$$

where *l* is the length of the deck and α' is the estimated

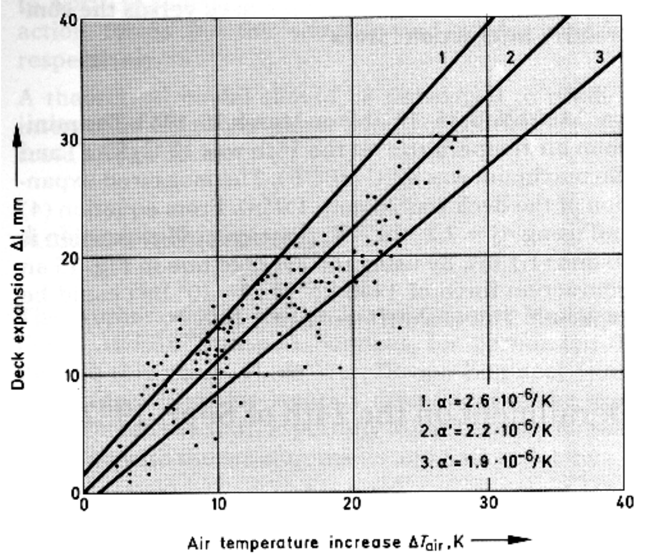

Fig. 14: Bridge deck expansion versus air temperature. The values of α' are possible "temperature coefficients" for this deck

"temperature coefficient" for this type of deck. The values of a' are given by:

$$\alpha_1' = \frac{1.30}{495{,}000} = 2.6 \cdot 10^{-6} \tag{5}$$

$$\alpha_2' = \frac{1.10}{495{,}000} = 2.2 \cdot 10^{-6} \tag{6}$$

$$\alpha_3' = \frac{0.95}{495{,}000} = 1.95 \cdot 10^{-6} \tag{7}$$

In **Fig. 15** the daily expansion of the deck is plotted against the compressive interaction force. Most of the points are situated in a band. Knowing the maximum difference in air temperature, the corresponding expansion of the 495 m (1624 ft) deck or any other length can be calculated by using equation (4). From **Fig. 15** the compressive interaction force can be found. The maximum compressive interaction force recorded was 1490 kN ($3.35 \cdot 10^5$ lbf) on March 16,1983. The minimum air temperature on the 16th was 15°C (59°F) and the maximum was 42°C (108°F). The measured expansion of the deck was 26 mm (1.0 in). From equation (4) and using $\alpha'_2 = 2.2 \cdot 10^{-6}$/K, the expected expansion is 29 mm (1.1 in). By using the average line in Fig. 15 an interaction force of 1440 kN ($3.24 \cdot 10^5$ lbf) could be expected. This is a difference of 3.4 %.

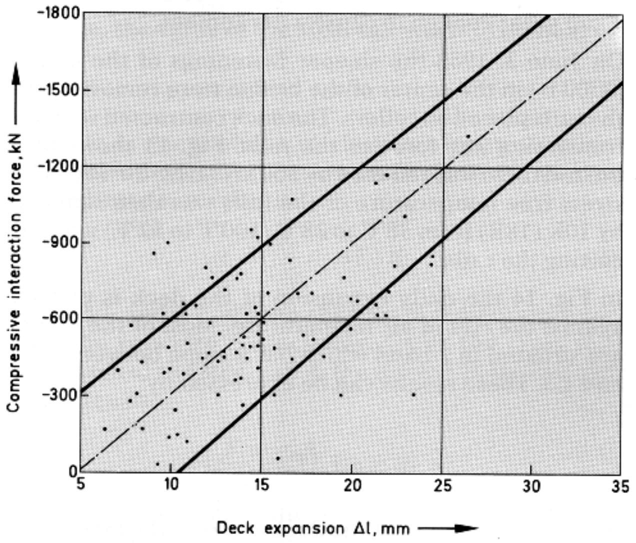

Fig. 15: Expansion of the bridge deck versus the compressive interaction forces

Derailment on the 24th of Sept. 1982

In order to reconstruct the derailment which occured in the center of the bridge on the 24th of Sept. 1982, 4:45 p.m., the conditions existing at this time must be determined. Temperatures in the vicinity of the bridge were obtained from a station of the Weather Bureau 4.0 km (2.5 mi) away from the bridge. Most of the seasonal change in weather had occurred in the four days prior to the derailment. The maximum temperature rose by 18K (32R) in this period. On the day of the derailment the temperature rose from 10°C to 40.5°C (50°F to 105°F), a change of 30.5K (55R). From equation (4), with $\alpha'_2 = 2.2 \cdot 10^{-6}$/K follows an expected expansion of 33 mm (1.3 in) and from Fig. 15, an average interaction force F_i = -1750 kN ($3.93 \cdot 10^5$ lbf).

On October 22, 1982 similar temperatures as on the 24th of September were recorded. At 5: 00 p.m. the air temperature was still 40°C (104°F). From this the maximum rail temperature during the derailment was estimated at 58°C (136°F). The temperature force F_T in two rails can be calculated with the following formula:

$$F_T = 2\,\alpha_s A_s E_s (T_o - T_s) \tag{8}$$

Where the thermal expansion coefficient of steel α_S is $11.5 \cdot 10^{-6}$/K ($6.39 \cdot 10^{-6}$/R), the cross sectional area A_S of a 60 kg/m (40 lb/ft) rail is 7600 mm^2 ($8.18 \cdot 10^{-2}$ ft^2), the modulus of elasticity E_S of steel is 205000 N/mm^2 ($29.7 \cdot 10^6$ lbf/in^2), the destressing temperature T_0 valid during this time period was 30°C (86°F), the rail temperature T_S is 58°C (136°F). Therefore, F_T = -1004 kN ($2.26 \cdot 10^5$ lbf).

Assuming 70 % of the long term interaction force F_{lt} as shown in **Fig. 10c**, an interaction force F_i = -1750 kN, and a temperature force F_T = -1004 kN for two rails it then follows that the total force in the rails at the expansion joints could have been:

$$\begin{aligned} F_{tot} &= F_i + 0.7\,F_{lt} + F_T \qquad (9)\\ &= -1750 - 0.7 \cdot 1100 - 1004 \\ &= -3524\ \text{kN}\ (-7.9 \cdot 10^5\ \text{lbf}) \end{aligned}$$

Fig. 9b showed that the rails will move in relation to the bridge deck whenever the forces are greater than 7 kN/m (480 lbf/ft). The rails are laid down on the bridge deck in a linear fashion, and the major portion of this movement is in a longitudinal direction. When the rails are not perfectly aligned, and the compressive forces are great enough a lateral movement will also be seen. The alignment error *f is* the lateral deviation in mm from the ideal linear rail axis based on 1 m rail length.

When the compressive force in the rails is high enough to cause a lateral movement it can occur in the form of a double wave kickout of the rails. This critical compressive force F_{cr} can be calculated using the following equation:

$$F_{cr} = -2.95\left(\frac{E\,I\,q}{f}\right)^{\frac{1}{2}} \tag{10}$$

where the equivalent moment of inertia I of the rail about the vertical axis is $1.6 \cdot 10^{-5}$ m⁴ ($1.85 \cdot 10^{-3}$ ft⁴) and *f* is the ordinate of alignment error. In equation (10) *q* describes the resistance of a rail to lateral buckling. In this case the lateral resistance *q* was assumed to be about 5 kN/m (343 lbf/ft). From equation (10) it follows that the critical force for an alignment error of 20 mm (0.79 in) is

$$F_{cr} = -2671\ \text{kN}\ (6.0 \cdot 10^5\ \text{lbf}) \qquad (11)$$

And for an alignment error of 12 mm (0.47 in) we obtain

$$F_{cr} = -3450\ \text{kN}\ (7.7 \cdot 10^5\ \text{lbf}) \qquad (12)$$

On the day of the derailment, with a probable total force of - 3524 kN as calculated from equation (9), the track was just as likely to kick out with an alignment error of 12 mm as with 20 mm. In order to increase the lateral resistance of the rails to movement, heavier PY-wing sleepers were used. A wing sleeper has four projections or wings, two on each end, which provide extra surf ace area for lateral and longitudinal resistance to movement. The friction force increased to 10 kN/m (685 lbf/ft).

Conclusions

The derailment on September 24,1982 was caused by a kickout of the track.

From the use of strain gages and carrier frequency amplifiers for the electrical measurement of mechanical quantities, to provide accurate data, significant relationships between the temperature and the forces in the rails on the bridge were developed.

With a knowledge of the daily fluctuations in air temperature the interaction forces for similar decks can be predicted.

On the basis of these relationships a simple system can be developed for district engineers to determine when to destress the track, especially in places where there are problems with creep.

A splice joint in the middle of the bridge will lessen the problem of kickouts but will cause a weak spot in the structure. Such a splice joint would have to be special and should allow movement of 250 mm to 300 mm (9.8 to 11.8 in). A monitoring system is preferred.

A special monitor and alarm system to determine the high compressive and tensile forces was proposed to the System Civil Engineer. Levels should be set for the maximum compressive and tensile forces allowable in the rails at the expansion joints. If an alarm condition should occur it would be signaled to the control center in Saldanha. Such a scanning system based on the UPH 3200 was designed by HBM and SAT. It has the task of providing long term monitoring and registration of measured values and also of providing alarm signals to the control center at Saldanha if the forces in the rails exceed set limits. In this way the risk of further derailments has been greatly reduced. This design is special in that in addition to recording and providing alarm indication the system will also record which signal produced the alarm and its value. **Fig. 16** shows the UPH 3200 system and associated equipment after its installation during February 1984.

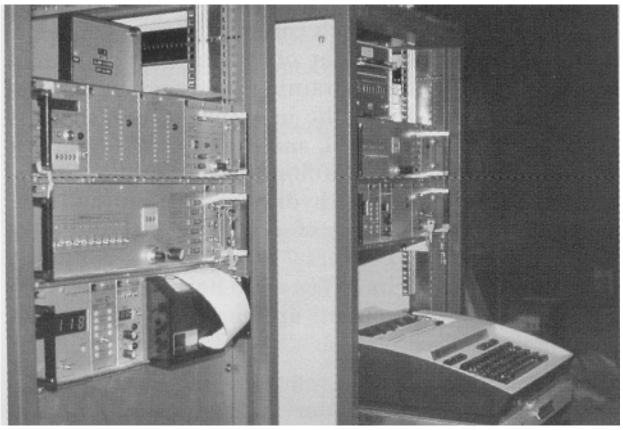

Fig. 16: UPH 3200 scanning system to provide long term monitoring and alarm signals

Recommendations

The lateral resistance of the track on the bridge can be increased 15 % or 20 % by increasing the ballast profile at the sleeper shoulders. This has been done.

The ballast thickness under the sleepers must not exceed 250 mm (9.8 in). This further increases the stability provided by the ballast retaining walls. The ballast thickness under the sleepers was reduced from 500 mm to 250 mm (19.7 to 9.8 in) shortly after the derailment.

PY-wing sleepers which provide greater lateral stability should replace the FY concrete sleepers. This was done during February 1984. The Austrian Railway found that lateral resistance increased more than 100 % with wing sleepers.

Tamping of the track should only occur during June and July, the winter months with low compressive interactive forces, thus giving a small probability for a kickout of the track.

Redistribution of the interaction forces by removing sleeper fastenings and placing the rails on rollers, should be done during August to September and March to April, to reduce the compressive and tensile interaction forces for the summer and winter months respectively.

A theoretical model should be developed to predict interaction forces per meter length of continuous concrete bridge decks.

Acknowledgements

The Author wishes to thank the General Manager, South African Transport Services, for permission to publish this paper. H. de Wet, Design Engineer from the Bridge Office for lengthly discussions of the problem and all the staff, especially Mike Tomas, who was involved with the measurements and data reduction.

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