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# Design and Construction of the Flood Resistant Pongola River Bridge

## Projet et construction du pont sur le Pongola

## Projekt und Bau einer Brücke über den Pongola

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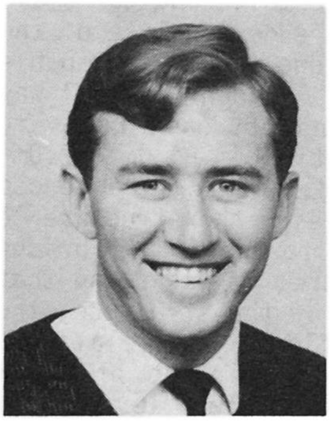

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**SUMMARY**

This paper describes briefly the design and construction of the Pongola River Bridge, Zululand, built to replace an earlier structure destroyed by cyclonic floods. The new bridge has been designed to resist overtopping by similar floods in the future. The loadings assumed to apply during flood conditions are described, as well as the measures taken to ensure that the structure can resist them.

**RÉSUMÉ**

L'article décrit le projet et la construction du pont sur le Pongola, Zoulouland, destiné à remplacer un ouvrage d'art détruit par une crue brutale. Le pont a été conçu pour résister à de semblables crues dues à des cyclones. Les cas de charges hypothétiques sont décrits, de même que les mesures prises en vue d'assurer la résistance de l'ouvrage.

**ZUSAMMENFASSUNG**

Dieser Artikel beschreibt das Projekt und den Bau einer Brücke über den Pongola, Zululand, welche ein Bauwerk ersetzt, das von durch Wirbelstürme ausgelöste Hochwasserfluten zerstört wurde. Die neue Brücke wurde so gebaut, dass sie allfälligen ähnlichen Hochwasserfluten standzuhalten vermag. Die angenommenen Lasten sowie die getroffenen Massnahmen, um den Widerstand des Bauwerkes zu gewährleisten, werden beschrieben.

## 1. BACKGROUND

In February 1984 tropical cyclone "Domoina" struck the coast of south-eastern Africa, causing extensive flooding and damage over parts of South Africa, Swaziland and Mozambique. One of the many structures damaged or destroyed was the Pongola River Bridge, in northern Zululand, where nine out of the original ten spans, together with over half of the substructure, were washed away by flood waters that overtopped the deck by about 3 m. The bridge formed a vital link between the industrial heartland of South Africa and the coast, and rapid replacement of the structure was essential.

## 2. CONCEPT OF REPLACEMENT STRUCTURE

The original deck soffit level was above the 200 year flood line, but, in view of the devastating effect on the road network of Cyclone Domoina and the small but real possibility of a similar event occurring during the design life of the replacement structure, the Owner required that the new bridge should be capable of resisting cyclonic floods of a similar order to Domoina. The conventional way of achieving this, by raising the level of the deck and approach roads to provide freeboard over flood waters would have cost an estimated R4,0m ($2.0m) including necessary realignment of approach roads. Instead, the Engineer chose to reconstruct the bridge at the original level and to design the structure to resist overtopping by cyclonic floods. Although the flood forces on the bridge due to hydrodynamic effects and debris impact are very severe, it was found that by paying due regard to the configuration and proportions of the structure, the additional cost of providing resistance to overtopping floods was relatively small. The replacement bridge was constructed for a contract price of R2,9m ($1,5m), scarcely more than a conventional structure at that level, and considerably less costly than the raised alternative.

## 3. THE DESIGN

### 3.1 Derivation and description

About one third of the original substructure remained after the flood and the economical use of this dictated the relatively short span lengths of 23,5 m. The requirement that the bridge should be able to resist overtopping by cyclonic floods encouraged the use of a shallow, heavy deck which would minimise both the hydrodynamic forces on the deck itself and the tendency for the deck to become bouyant. A continuous structure was also preferred to a series of simply-supported spans because of its better resistance to isolated debris impact loads.

These criteria led to the choice of an 11 span continuous, reinforced concrete, voided slab with cantilevers. A beam and slab configuration like the previous structure was rejected both for hydraulic reasons and because the replacement bridge is wider than the previous one, and this would have meant widening the piers to accommodate the additional beams. Both full and partial prestressing of the slab were investigated but were found to be considerably more costly than the reinforced concrete alternative. Circular voids were chosen in preference to rectangular ones to simplify construction of the shallow, 1,6 m deep slab. The general arrangement of the bridge is shown in Figure 1.

The capricious nature of the Pongola River during the summer months made a cast-in-situ method of construction using conventional falsework extremely unattractive. On the other hand, the incremental launch method of construction, as well as showing economic advantages, avoided any work on the superstructure taking place in the river. For these reasons the deck was constructed by incremental launching from the South bank.

### 3.2 Special characteristics

The construction of a reinforced concrete, voided slab deck by incremental launching - a method usually employed only with prestressed box-sections - made it necessary to consider a number of unusual factors in the design.

First, the deck was reinforced, not prestressed. The effect of this is most significant at the construction joints between segments. While, as is common practice, these were arranged to be at quarter-span positions in the final condition, during launching each joint is subjected cyclicly to high sagging and hogging moments. Without the customary axial post-tensioning force, the possibility of either shrinkage or flexural cracking at the joints has to be allowed for.

The second significant change from normal incremental launching practice was the use of a slab cross-section rather than a box. This has important implications at the interface between the steel launching nose and the concrete deck both because of the abrupt change of cross section there and the shallow depth of section relative to the moments carried.

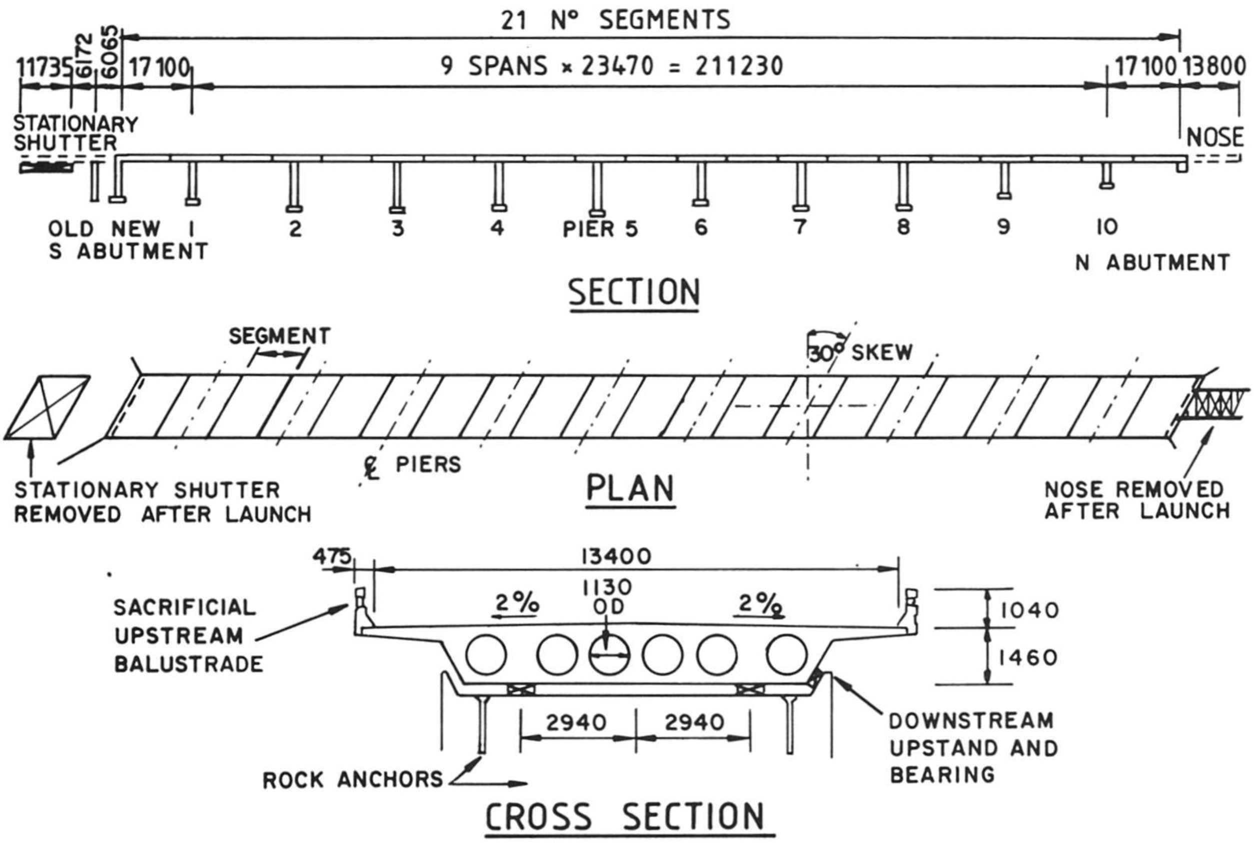



Figure 1: GENERAL ARRANGEMENT

In cross-section, the deck comprised 6 circular voids with, correspondingly, 7 webs. The two bearings were positioned under the webs adjacent to the outermost pair (that is, under webs 2 and 6). In the final condition, loads are transferred to the bearings by the heavy cross-beam located over the bearings, but clearly during the launch this cross-beam was not always present. Without the cross-beam, very high shear forces have to be transmitted transversely across the top and bottom flanges of the circular voids adjacent to the bearings. This effect was minimised by departing from the usual uniform void spacing and increasing the width of the two webs positioned on the line of the bearings, as shown in Figure 1.

Both the previous and the replacement bridge have a skew angle of 30°. Clearly, this moderately high skew angle had implications for the design of the deck, especially over the front portion and launching nose during the launching.

## 4. HYDROLOGICAL AND HYDRAULIC ASPECTS

### 4.1 Hydrology

The catchment area of the Pongola River above the bridge is 7060 km², and is partly bushveld and partly grass savannah in the upper regions. The catchment is entirely rural and has a mean annual precipitation of 960 mm. The Pongola has flooding characteristics typical of rivers in Southern Africa, where rainfall is concentrated in a few months of the year. Vegetation grows rapidly between flood occurrences and the amount of vegetative debris carried by typical floods is considerable.

The Domoina flood was, of course, exceptional. At its peak the discharge was estimated at 16 000 cumec, which may be compared with "normal" flood discharges for the Pongola of 1800 cumec for the 10 year return period flood, 4000 cumec for the 100 year and 5000 cumec for the 200 year flood. During the flood, waves 3 m high were reported by eye-witnesses, while the river, normally less than 50 m wide, was over 300 m across. After the floodwaters had subsided, masses of debris up to 5 m wide were found lodged on the remaining piers and deck span of the original bridge.

The Domoina flood has had a significant effect on the future hydrological response of the Pongola River. The discharges and velocities of flow resulting from rainfalls of given return periods are much higher now than before and this had to be taken into account in designing a replacement structure to resist future floods.

The 200 year return period flood was selected as the design flood which defined the vertical alignment of the bridge. It was considered that providing 1 m freeboard over this flood would provide the best compromise between the two traditional approaches, the low cost, low level causeway and the high cost, high level bridge. At Pongola, a lower vertical alignment would have yielded no cost advantage, because of the existing approach roads. However, in other circumstances a lower alignment may be preferable, down to, say, the 20 year raturn period flood level. The choice of flood that could be passed before overtopping occurs would depend on the strategic and economic importance of the route and the cost of delays during flooding.

### 4.2 Flood design parameters

In designing the bridge to resist overtopping by cyclonic floods, loading parameters and combinations were to some extent empirically derived from observations of the Domoina floods. Conventional hydraulics and wave action theories have limitations when applied to conditions as unpredictable and subject to such rapid fluctuations as in this case.

A water velocity of 5 m/s was estimated from hydraulic data and was used to assess hydrodynamic effects. To simulate the indeterminate effect of wave action during the critical period just as the bridge deck is overtopped an upward component of water velocity on the underside of the deck of 2 m/s was adopted. When assessing hydrodynamic and hydrostatic effects the projected pier widths and deck depth were taken as 5 m to represent accummulated debris. Some design codes require the application of a single, large force on the structure to represent the impact of an isolated large item of debris, such as a large tree. However, with a continuous deck this becomes a superfluous loading whose only effect would be local damage, which is in any case acceptable during overtopping.

Three flood load combinations were considered, representing three stages which were assumed to occur as the flood waters rise and the bridge is overtopped, viz:

Combination 1 : This occurs as the water reaches the soffit of the deck, and is shown in Figure 2(a). The deck still has its full self weight, that is, hydrostatic bouyancy forces are not applied. Debris is assumed to collect on the upstream edge of the deck to a height of 1,5 m above road level, but not below soffit level, since this would reduce wave action.

Combination 2 : This covers possible hydrostatic effects as the water dams up behind the deck and accumulated debris just prior to overtopping. See figure 2(b). The deck still has its full self weight, but is subjected to hydrostatic uplift on the upstream side of the soffit from the 2 m head of water created by the debris. This is a severe loading which is extremely unlikely to occur over more than a small proportion of the length of the deck at one time.

Combination 3 : This is the fully submerged condition. The deck is subjected to full hydrodynamic loading from the flow acting on a 5 m depth of debris, positioned centrally on the upstream edge of the deck. The deck has its submerged self weight, which includes hydrostatic bouyancy forces.

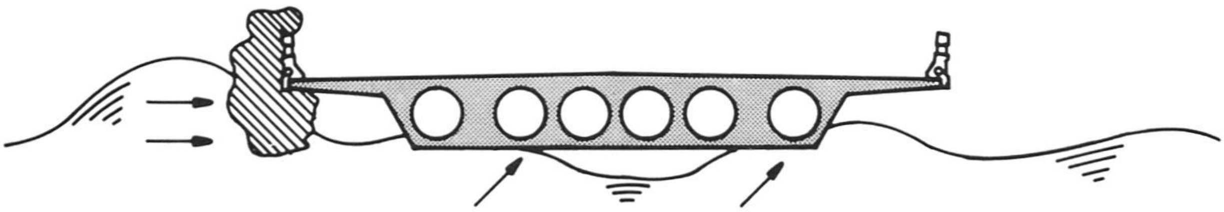
In analysing the stability of the deck, the entire 240 m length was assumed to be subjected equally to the above loadings. This is a conservative approach, especially with such highly variable actions as wave forces, but allows the analysis to be carried out per unit length of deck. It can be shown that any torsional or flexural moments induced in the deck by variations in conditions along its length are low relative to design traffic load effects.

Hydrodynamic forces, P, were calculated from the common relationship:

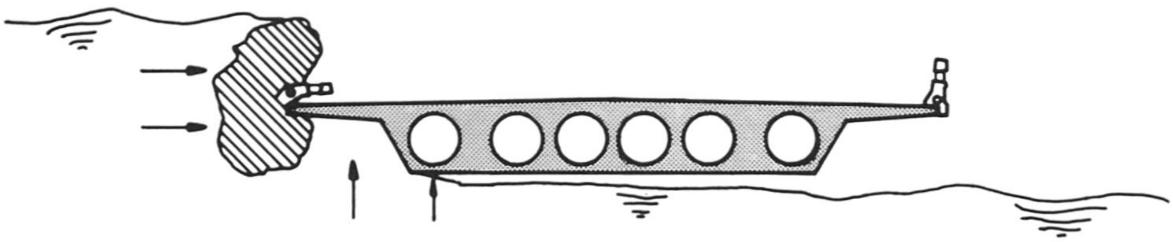
$$P = v^2 k A$$

where v is the water velocity, k is coefficient related to the shape of the body and A is the cross-sectional area at right angles to the direction of flow. The value of k used was 0,7, corresponding to the irregular shapes caused by the collected debris.

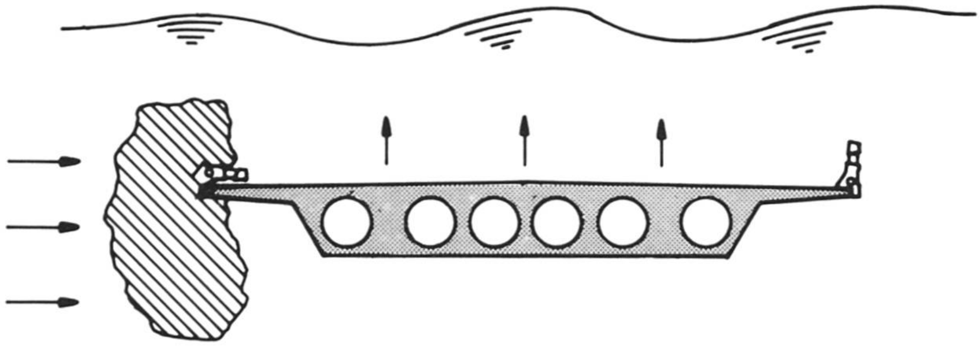
The flood loadings were taken to be realistic working loads, not ultimate loads, and therefore an overall minimum factor of safety against instability of 1,50 was provided in the case of load combinations 1 & 3. A reduced value of 1,25 was used for combination 2 to reflect its lower probability of occurrence along a significant proportion of the length of the deck at any one time.



(a) Stage 1: Hydrodynamic forces on accumulated debris and wave action on deck.



(b) Stage 2: Horizontal and uplift hydrostatic forces — deck & debris acting as dam.



(c) Stage 3: Hydrodynamic forces on fully submerged deck and accumulated debris.

Figure 2: LOADINGS CONSIDERED IN DESIGN OF BRIDGE AGAINST OVERTOPPING FLOODS.

When assessing pier stability, the above load combinations were applied to the superstructure, together with hydrodynamic forces acting on a 5 m width of debris collected on the pier over its full height. It is common practice, when designing bridge piers against flood forces, to include the effects of hydrodynamic "lift" forces acting at right angles to the direction of flow, caused by the flow running at an angle to the pier. This is appropriate to, say, European rivers, but the large amounts of debris carried by African rivers in flood act to break up the streamline flow around the piers and reduce the "lift" effects. Nevertheless, the piers were also checked for this effect, but without debris in place.

### 4.3 Resistance to flood loadings

To resist these very severe but feasible loadings, the primary requirement was to ensure stability of the existing and partially reconstructed piers, which were founded on spread footings on hard dolerite and mudstone. This was achieved by installing rock anchors through the full height of each pier and stressing from the new pier heads. The stability of the deck on the piers was ensured by careful choice of deck type and configuration. A continuous deck was chosen, in place of simply supported spans, since this provides better distribution of flood forces between piers and thus allows the deck to remain stable under isolated high forces due to waves or debris impact. Bouyancy of the deck is also a vital factor, and by adopting a relatively heavy deck cross-section, the hydrodynamic and hydrostatic uplift forces can be resisted without the need for expensive holding down bolts or uplift bearings. A beam and slab type of deck is not a good choice if the bridge risks being overtopped, because of the bouyancy of the trapped air between the beams and the wave action on the beams. To provide the necessary horizontal restraint during floods, each pier has a substantial upstand on the downstream side and only standard sliding bearings are required to support the deck during normal service. Finally, the New Jersey-type balustrade on the upstream side is designed to resist full traffic impact loading from the roadway but to collapse in the downstream direction under the overtopping flood loading. By this device, the projected area of the deck is reduced and the resulting deck cross-section offers low disruption to flow and should minimise the tendency of debris to collect on the upstream edge.

The approach earthworks on the South bank are partly protected by the abutment breast and wing walls, but during an overtopping flood it was accepted that some damage to the approaches would occur. On the North bank, the approach is in cut and the amount of fill that would be vulnerable is in any case small. As all the foundations are spread footings onto rock, scour cannot occur.

## 5. CONSTRUCTION

A certain amount of demolition was necessary as the first task for the reconstruction of the bridge, although most of the work had been carried out very efficiently by Cyclone Domoina. One span remained after the flood, but it was badly damaged and was removed. About 30% of the original substructure remained. The abutments were demolished where they interfered with new construction. The piers were originally of unreinforced concrete and remained intact to varying heights, and these were re-built to the required level with lightly reinforced concrete. The rock anchors were then installed from the pier tops in order to introduce a small prestress into the concrete as well as to secure them against flood forces. Sleeves were cast into the new concrete and the anchor holes drilled through these, through the existing concrete and into the rock.

The deck was then constructed by incremental launching from the South bank. The lack of prestress and overall simplicity of the deck cross-section allowed for the unusually short cycle time for the preparation, casting and launching of each segment of 5 days. Segments were launched after about 62 hours, once a concrete strength of 15 MPa had been obtained. This is considerably less than the usual strength requirement for an incrementally launched deck. With the 5 day cycle, the bridge deck was constructed at a rate equivalent to 33 m$^2$ of deck per day.

The bridge is shown under construction in Figure 3.

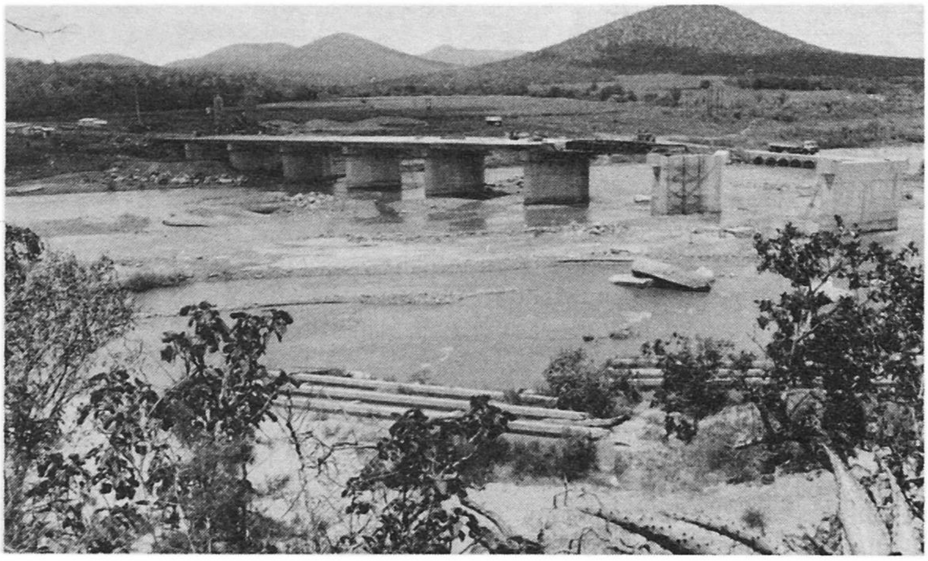


Fig. 3 : Bridge during construction, with remains of earlier structure in the foreground.

## 6. CONCLUSIONS

The Pongola River Bridge demonstrates that by careful choice of bridge type and configuration it is possible to produce a structure that, while ostensibly conventional, is able to resist the extremes of nature for negligible additional cost.

The approach used at Pongola could undoubtedly be employed with benefit elsewhere, especially in those parts of the world with intemperate climates. As always, however, the choice of optimum solution depends on the particular situation. For example, the shape of the river valley cross-section may affect the choice of vertical alignment of the road. Similarly, the importance of the route - strategic or economic - would dictate the choice of flood to be resisted before overtopping occurred. Clearly, a less vital link than Pongola could be located above, say, the 20 year flood line instead of the 200 year level.

The Pongola River Bridge was completed in a contract period of 12 months, which is short, even allowing for the relatively small amount of substructure construction. An important contributory factor to this achievement was the simplicity of the design which provided for simplicity of construction. Even today, the importance of this aspect is all too often overlooked.

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