The Collapse of the K-B Bridge in 1996
A MSc Dissertation at the
Imperial College London

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Introduction

I had studied civil engineering combined with business studies at the University of Leipzig since 1993. After 3 years in Leipzig I went to the Imperial College of Science, Technology and Medicine in London to attend the Master of Science course "concrete structures". The course was designed for civil and structural engineers to develop their skills in analysis and design in the context of concrete materials and structures.

The reasons for choosing this course were on the one hand the very good reputation of the course and my interests in this field and on the other hand the wide selection of cultural life in London.

In the last trimester of the course I wrote a final dissertation under supervision of Prof. England about the collapse of the Koror - Babeldaob Bridge in the Republic of Palau. The paper provides general information about the bridge and the collapse being available for the public at this time and includes a stress and deflection analysis for the most important time steps. Then the longitudinal stresses at the time of the collapse are calculated.

The analysis of the structure was partly based on assumptions since not all information needed was accessible. Hence the results could only indicate probable regions of stresses which might have led to the collapse.

The following pages give some general information and results of the research. More detailed information is given in [2] and [3].

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The Koror - Babeldaob (K-B) Bridge was situated in the Republic of Palau, about 1200 km north of New Guinea and 850 km southeast of the Philippines in the Western Pacific. It linked the two islands of Koror and Babeldaob and replaced a cable guided ferry.

1 ORIGINAL DESIGN

The bridge was designed in 1975 by Alfred A. Yee and Associates, Inc., Honolulu. At the time of the finish of construction in 1977 it was the prestressed concrete box girder cantilever bridge with the longest span in the world..

The general layout of the bridge is shown in figure 1. The main span was 240.8 m long. The side spans which had been chosen for aesthetic reasons were 53.6 m long. From the end piers cantilevers of 18.6 m extended towards the abutments. So the overall structure was 385.20 m long.

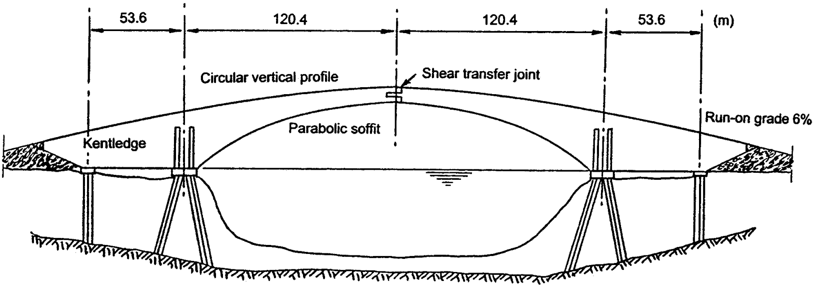

Fig. 1: General Layout Details

Built up earth embankments of about 150 m length provided the elevated approaches. The approach roadway had a 6 % grade and was rounded by a 243.80 m vertical curve which was symmetrical about the centre line of the bridge.

A typical cross section of the cantilever is shown in figure 2. At the central hinge the box girder was 3.66 m high and increased parabolically to 14.17 m at the main piers. Then it reduced linearly in the side spans to 10.26 m at the end piers and tapered sharply to 2.74 m at the abutment.

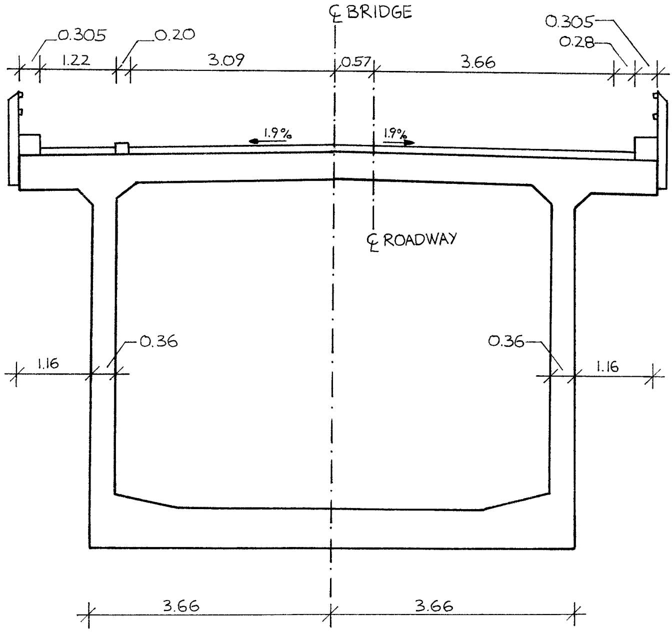

Fig. 2: Typical Cross Section

The end spans of the structure were partly filled with crushed rock to form ballast compartments in order to balance the cantilevers. The end piers had sliding bearings to allow free longitudinal movement and a tie-down feature to balance the maximum overturning moment of the structure. The superstructure was monolithically connected to the main piers which had piled foundations. This made it possible to support unbalanced moments at the main piers which occurred during the construction process.

The central hinge was necessary to enable movements due to creep, shrinkage and thermal changes. It was of a tongue-and-groove type and could only transfer vertical and lateral shear forces from superimposed loads.

Hence the cantilever structure was statically indetermined to 3rd degree. However, under symmetrical vertical load (i.e. self weight) the cantilever arms behaved in a statically determined way.

The layout of the longitudinal tendons was approximately triangular, because of the cantilever type of the structure. Therefore the number of tendons to be anchored was almost the same at each segment. Near the main piers the top slab had to be thickened to accommodate the large number of 316 tendons which were arranged in four layers. The thickness decreased as the number of tendons decreased. At the main piers the longitudinal prestressing force was 182.4 MN.

First of all the bending moments due to the dead load of the structure were calculated with a self written computer program. The analysis showed that the calculated prestressing forces compensated the bending moments and led to an uniform compressive stress distribution at the end of the segments. This meant that the initial displacements were only due to longitudinal shortening of the cantilevers without vertical deflections.

Later, because of the reducing prestress, downward deflections would occur and increase with time as the stress distributions over the depth of the cantilevers became less uniform, with higher compressive stresses developing in the soffit slab.

This state developed fairly quickly in the K-B Bridge because the roadway which was constructed afterwards led to negative bending moments. The resulting stresses in the top and bottom edge of the cross-section are shown in figure 3 for the entire cantilever. These stresses were positive (compression) everywhere and in the allowable range.

The differences between the stresses in the top and bottom edge increased with time due to creep, shrinkage and relaxation and led to larger deflections over the years.

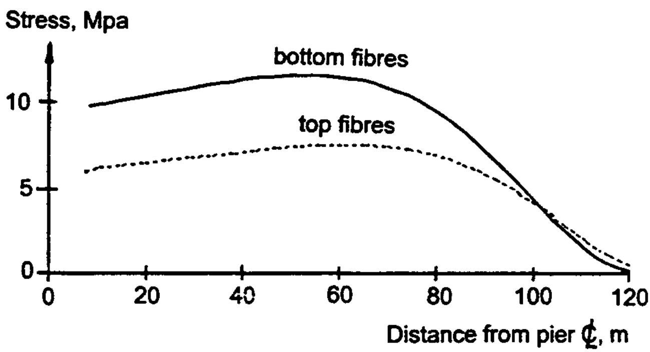

Fig. 3: Initial Stress Distribution At The End Of Construction Phase

2 SAFETY AND DEFLECTION STUDIES

Like many other bridges built in the free cantilever method the K-B Bridge suffered unacceptable time-dependent deflections during its early service life. The central span of the bridge deflected approximately 1.20 m until 1996. This led to two studies carried out by Louis Berger International and the Japan International Co-operation Agency. They concluded that the bridge was safe and the large deflections were due to creep and the modulus of elasticity of the concrete in place being lower than anticipated.

In 1993 a third study was done by ABAM Engineers, Seattle. They estimated that the deflection would increase by another 0.84 m in the next 85 years and recommended a strengthening of the bridge by additional external post tensioning. The design required 40 external post-tensioned tendons and maintained the original cantilever design.

The creep and shrinkage of the concrete and the relaxation of the steel was modelled using the CEB-FIP Model Code from 1978 which is based on the Rüsch-Jungwirth Method. This led to creep coefficients between 2.25 and 2.42, strains due to shrinkage of about $190*10^{-6}$ and a loss of 15% of the initial prestress.

These time dependent effects induced a reduction in positive moments created by the prestress which led to lower compressive (or small tensile) stresses in the upper edge and high compressive stresses in the lower edge. The new concrete stresses which were still in the allowable range are shown in figure 4. The results are subject to slight variation depending on the way how creep and shrinkage strains are evaluated and allowed for the analysis.

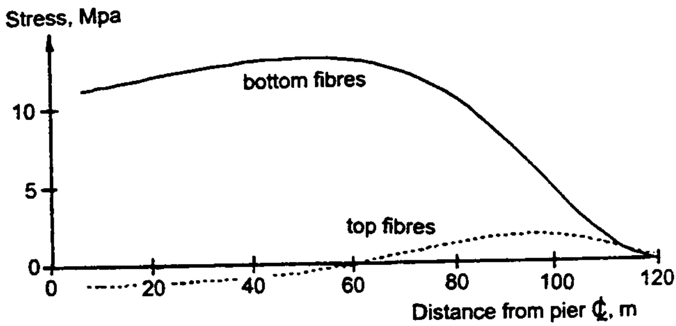

Fig. 4: Estimated Edge Stresses Before The Retrofit

The small tensile stresses in the top edge might be attributed to the rough modelling of the creep behaviour or indicate tensile cracking in the top deck. Cracks would explain the large deflections of 1.20 m at midspan which could not be achieved by the deflection analysis. The analysis showed 0.46 m as value of deflection.

For the further analysis uncracked section properties were used and it was assumed that the small tensile stresses had no influence on the behaviour of the structure.

3 REMEDIAL WORKS

The K-B Bridge was repaired between 1995 and 1996 following an alternative proposal of the post tensioning contractor VSL. It was suggested to install 8 external post-tensioning tendons continuous across the main span hinge with a total tensile force of 35.6 MN. This converted the balanced cantilever structure to a continuous one by connecting the two halves.

The stress reversal at the ends of the cantilevers after the locking operation would be balanced by deviating the tendons down to the bottom of the box at the centre, where they would pass through slotted holes cut into the joint walls.

In order to solve the problem with the foundation, VSL split the function of inducing extra compressive forces in the top flange between the tendons and a series of 12 flat jacks inserted in the central joint. The flat jacks with a total force of 31 MN were to assist the upward deflection and counteract the effect of the new post-tensioning forces being redistributed into the pile foundation substructure.

The jacking operation induced additional compressive axial forces and positive bending moments in the structure. The external tendons caused forces upwards near midspan and downwards approximately 67 m from the main piers (see figure 5).

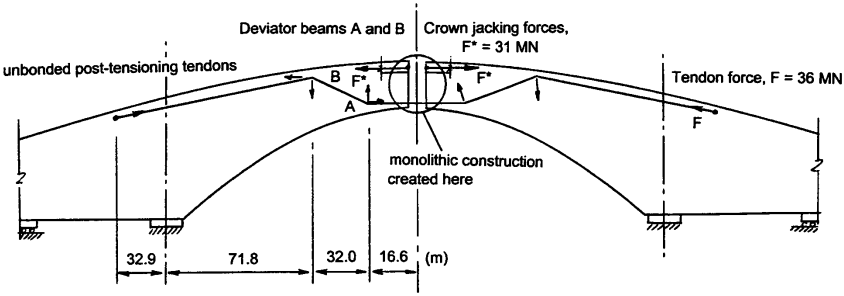

Fig. 5: VSL Value Engineering Proposal

The new stress distribution is shown in figure 6. It can be seen that the maximum compressive stress in the top slab was near the point where the downward force was applied to the structure. In the bottom slab the inverse behaviour led to maximum stresses near the main piers and minimum stresses at midspan. It can also be seen from the figure that the calculated stresses were in the allowable range.

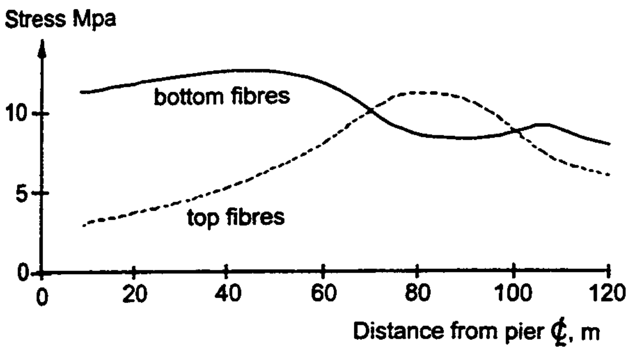

Fig. 6: Stresses In Concrete After Retrofit

4 COLLAPSE AND INVESTIGATIONS

On September 26, 1996, three month after the retrofit was completed, the K-B Bridge suddenly fully collapsed without any warning.

First investigations were carried out shortly after the collapse by SSFM Engineers, Inc., Honolulu for the United States Army Corps. They said that the main span was a total loss as most of the concrete structure rests on the bottom of the channel. They suspected that a chain reaction failure had occurred, starting at the top deck near the main pier on the Babeldaob side. This slab was completely destroyed and might have lost its capacity to resist the compressive forces from the prestressing. A summary of the damage of the bridge as identified by SSFM is given in figure 7. Due to a lack of redundancy for this type of structure, the bridge might have collapsed during some seconds after occurrence of the failure of the top deck slab.

In the following months engineers from all over the world developed their theories of the collapse. In general, most of them explained the reason of the collapse in the change of the structural system and the effects it had on the structure.

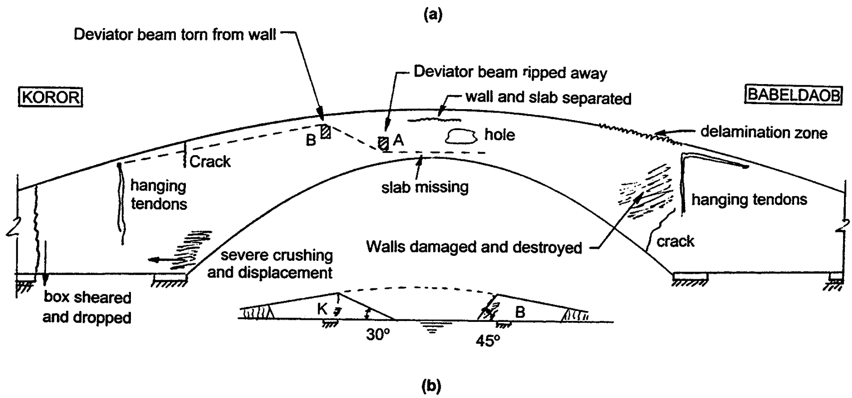

Fig. 7: Summary Of Damage As Identified By SSFM

5 DISCUSSIONS AND CONCLUSIONS

The analysis provides stress distributions at each important stage during the life of the bridge. The calculated stresses never exceeded the allowable values. It can be concluded that the collapse of the bridge was probably not due to excessive stresses, caused by axial forces and bending moments. Even assuming a low concrete quality would not lead to a collapse. Especially a primary compression failure at the support as supposed by SSFM seems not possible.

This can be supported by a simple consideration: Assuming a linear stress distribution and a compressive strength of 35 MN/m^2 the minimum force required to cause a compression failure at the top before a tensile failure at the bottom is 366 MN. Such high compressive forces could not be calculated and are very unlikely. In the same way a minimum positive bending moment can be calculated as 1700 MNm which seems also not to be very probable.

But it has to be considered that the calculations were based on assumptions which must be discussed more in detail.

In the calculations it was assumed that the section was not cracked. A cracked section would possibly not be able to resist the compressive stresses after the retrofit. These cracks could be due to the tensile stresses before the strengthening program or external effects like a damage inflicted on the deck during the retrofitting operation.

In many papers the influence of temperature after closing the hinge is discussed. Temperature variations between June and September are very small (less than 1 K on average) in Palau and can therefore be excluded. It is unlikely that in 1996 the variations were much higher than the average variations.

More likely is the possibility that daily temperature variations caused the damage (e.g. closing of the hinge at night). In case of temperature variations large compressive stresses would be induced to the structure.

Other possible reasons for the damage are a bad alignment of the jacks in horizontal direction, small horizontal movements of the piers towards the channel or another modelling of the angles of the external tendons.

Also a shear failure of the structure was not considered. It was often discussed in the literature. The new road and the external tendons induced additional shear forces in the structure which were only partly compensated by the additional axial compressive stresses. Moreover these compressive stresses decreased with time due to the time dependent effects.

Time dependent changes of stresses between the retrofit and the collapse were not calculated because of the lack of information. They were mainly caused by the additional loads and the change of the static system.

In the calculations a linear behaviour was assumed. A more sophisticated analysis should take into account the non-linear effects. It was also neglected that the bridge was stressed in all directions. Also the contribution of the transverse prestress to the collapse was not considered.

Whatever the reasons for this collapse were, engineers should learn from it and use the gained information for new designs. Unfortunately it is possible that the primary cause may never be determined or published.

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