Why did Palau Bridge collapse?

Synopsis

The collapse of the Palau Bridge in 1996 received considerable attention at the time, but there has been very little reported in the literature about the investigation of the collapse mechanism, partly because of a legal agreement between the parties involved. This paper has been prepared from publicly available sources to ensure that the wider structural engineering community learns something from the failure. Since the collapse occurred soon after a repair to the bridge, it has been widely assumed that the repair was the cause of the failure, but it is shown that this is very unlikely. Instead it is concluded that lack of robustness in the original design meant that the structure had always been vulnerable to accidental damage, which eventually occurred as part of the resurfacing works.

Introduction

The failure of the Koror–Babelthuap Bridge in Palau, Fig 1, occurred on 26 September 1996, at around 5:45 in the afternoon[1]. The collapse was catastrophic, killing two people and injuring four more, and occurred under virtually no traffic load during benign weather conditions. Services passing through the bridge between the country's two most populated islands were severed; this caused the government to declare a state of national emergency and request international aid for the thousands of people left without fresh water or electricity.

In the 9 years since the collapse, there has been speculation regarding possible causes of failure, and remedies that could have avoided it[2,3,4,5,6,7]. Litigation and out-of-court settlements between the Palauan government and the engineers involved have meant that the cause was never officially confirmed, and analysis performed on site was never released. Only one paper has been presented based on the site investigations[8] and the true cause remains unreported.

The contrast with the aeronautical industry is marked. For even minor mishaps involving aircraft, international law requires a complete investigation with the publication of all reports, and there is widespread voluntary reporting of dangerous situations which did not lead to accidents.

In the structural engineering world no such requirement exists and, perhaps because the failure occurred in a far-away country, of which we know nothing (to paraphrase Chamberlain[†]), it has slipped from our consciousness.

The present study was undertaken to ascertain whether there is something fundamentally wrong with the way prestressed concrete is understood, and in particular whether it should be taught differently in the light of what happened. The authors are not associated in any way with any of the companies involved and have had no access to any confidential information; everything presented here has been derived using information already in the public domain. The objective has been to undertake simple approximate analyses to determine the magnitude of various effects that might have happened.

Palau

The name Palau (or Belau) refers to a group of about 350 small islands (centred at about 134° 30′ E and 7° 30′ N) at the western end of the Caroline chain in the Western Pacific. The islands are about 900km equidistant from the Philippines and New Guinea. Palau passed from German control to that of Japan after WWI; the chief industry then was the exploitation of phosphate deposits. The islands were of strategic importance to Japan in WWII, especially after the fall of the naval anchorage at Truk. A fierce battle was fought for one of the islands (Peleliu) in September 1944 but most of the islands were left in Japanese hands (although blockaded), until the general surrender at the end of the war. The islands remained part of the US Trust Territory for the Pacific Islands but are now independent, although they retain close ties with the USA. The total land area is only 494km^2 and the population today about 20 000[9]. The economy relies almost completely on tourism.

† 'How horrible, fantastic, incredible it is that we should be digging trenches and trying on gas masks here because of a quarrel in a far-away country between people of whom we know nothing' (Broadcast 27 Sept 1938).

Koror-Babelthaup Bridge

The bridge was designed to meet the need for a link between the two major islands of Palau; Koror and Babelthuap. The latter contained the country's international airport and was the source of most fresh water but approximately 70% of the population lived on Koror, where the capital is situated. The channel between the two islands is about 30m deep with tidal flows of up to 3m/s and steep banks, which is why a single 240m span (then the longest concrete girder bridge in the world) was chosen[10,11].

The original design was symmetric, each side consisting of a 'main pier' on the channel edge, from which cantilevers extended over the water, meeting in the centre. Outside the main piers were the approach spans, which rested on the 'end piers'. The main pier was supported on inclined piles which resisted horizontal forces, while the end piers had only vertical piles which will be assumed here to provide only a simple support. The cantilevers themselves, which were segmental and cast in place, were joined by a central hinge, intended to carry negligible load but to ensure displacement compatibility across the span. The hinge contained bearings to allow longitudinal movement and rotation of the half-spans relative to one another. An elevation of the original design is sketched in Fig 2. A 'box' cross section was used throughout, with fixed widths but varying depth, as shown in Fig 3.

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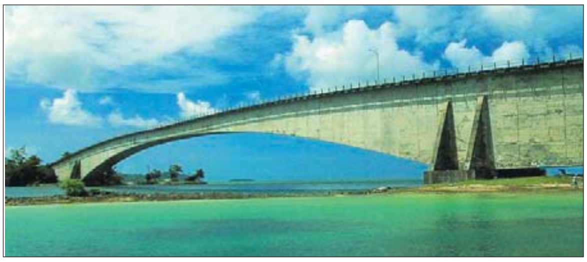


Fig 1. (left) The Koror-Babelthuap Bridge prior to collapse[22] *(photo: © William E. Perryclear)*

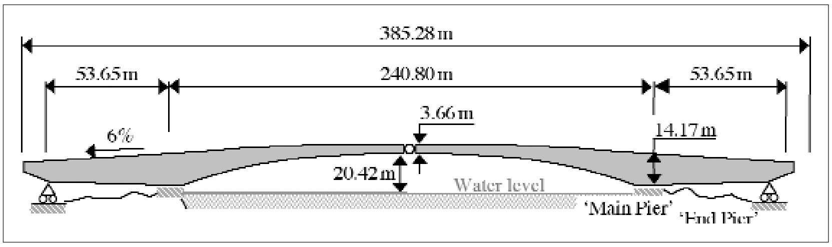


Fig 2. (below) Elevation showing bridge geometry, as built

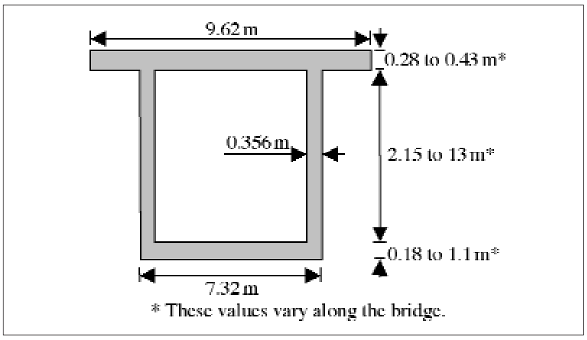


Fig 3. (left) Simplified cross section

Fig 4. (below VR model of original construction sequence (The model can be downloaded from www-civ.eng.cam.ac.uk/cjb/4d8/public/palau.html)

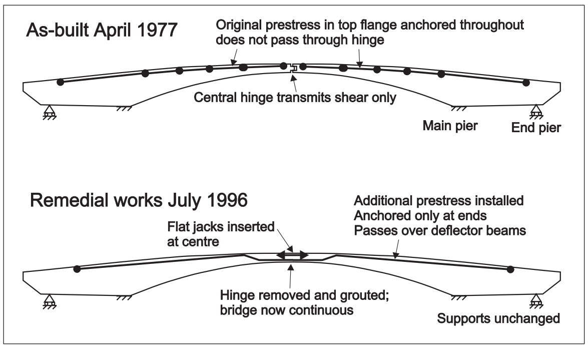


Fig 5. (left) Key features of original structural design and alterations made during remedial works

Fig 6. (above & right) Photos showing the bridge after the collapse[22] (a) the whole span viewed from the Babelthuap side (b) Koror side (West) (c) Babelthuap side (East) ***(photo: © William E. Perryclear)***

Each half of the bridge had been built as balanced but unsymmetric cantilevers, working away from the main pier, until the back span reached the end pier (Fig 4). This span was then filled with ballast to provide moment reaction for completion of the cantilevers.

Each side of the main span was prestressed using 316 Dywidag Threadbars (32mm diameter), with a total of 182.4MN of force anchored in the back-span between the piers. The other ends of the bars were anchored throughout the main span, at the ends of the 25 segments that made up each cantilever (Fig 5). In this way a smaller force was applied at the centre than at the piers, where a larger moment was experienced. This will be referred to as the original prestress to distinguish it from subsequent additions.

The bridge cost $5.2M to build, and was completed in April 1977, after which it remained unchanged for the next 18 years. Over this period the cantilevers deflected due to creep, shrinkage and prestress loss. By 1990 the sag of the centre line had reached 1.2m (visible in Fig 1), affecting the appearance of the bridge, causing discomfort to road users, and damage to the wearing surface. The Palauan government commissioned two teams of experts to assess the safety of the structure and its ability to continue to carry the design loads in the future. Louis Berger International (USA) and the Japan International Cooperation Agency both concluded that the bridge was safe and would remain so, but the deflection could be expected to increase further in the future (by another 0.9m over the next 100 years). As a result, the decision was made to put out to tender remediation works to correct some of the sag and prevent further deflection[8].

As part of the assessment of the bridge a loaded truck weighing 125kN was driven onto the tip of each cantilever to determine its stiffness[8]. The measured deflection was 30.5 mm, which corresponds to a Young's modulus of the concrete of $18kN/mm^2$.

A design proposed by VSL International[12] was accepted, and construction was carried out by Black Micro (a local firm). There were four elements to this 'retrofit' (shown diagrammatically in Fig 5):

- Removal of the central hinge to make the structure continuous
- Installation of eight additional, external, post-tensioned prestressing cables inside the box section, running beneath the top slab near the main pier and, via two deviator beams on each side, moving to the bottom of the box near the centre. All these additional tendons were continuous through the bridge, being anchored between the piers on each side. 36MN of force was applied to these cables, creating a hogging central moment intended to remove 0.3m of the deflection.
- Insertion of eight flat-jacks between the top slabs, in place of the central hinge, which were used to apply an additional 31MN of longitudinal compressive force. These were grouted in place, making the span continuous. The combined effect of the external cables and flat-jacks will be referred to here as the additional prestress. The decision to make the bridge continuous was a late amendment to the design, and apparently taken on economic grounds since it allowed the new cables to pass from one half-span to the other and thus halved the number of anchorages required.
- Replacement of the bridge surface throughout. Because the prestress would not eliminate all the sag, a lightweight void former was to be inserted over the central area under the new surface, to provide a smooth running surface.

The remedial works were completed in July 1996 and the surface replacement, performed by Socio Construction Company, finished in mid-August.

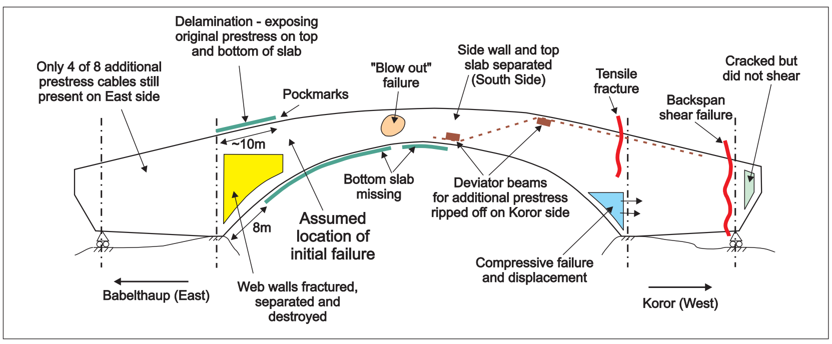
The collapse

The bridge collapsed on 26 September 1996. A report prepared by SSFM for the US army[13] describes in detail the most likely mechanism of collapse, inferred from eye-witness accounts and from visible damage to the bridge both above and below the waterline. Fig 6 shows pictures of parts of the bridge after collapse, in particular the area by the main piers on either side; a summary of the damaged regions is shown in Fig 7. The 'pockmarks' indicated in the figure were not mentioned in the SSFM report, but were discovered during later failure analysis.

The SSFM report describes the most probable mechanism of collapse as follows:

1. Delamination of the top flange occurred near the main pier on the Babelthuap side. This 'rendered it incapable of providing resistance against the original post-tensioning forces … causing the rest of the girder to behave as a reinforced concrete girder spanning between the [centre] and the [Babelthuap] main pier'.
2. Large hogging moments resulted over the main pier, inducing far greater tensile stresses in the top slab and upper region of the webs than could be sustained. The webs therefore failed at the top, resulting in near total loss of their shear capacity. As a result, the Babelthuap side of the span failed in shear, next to the main pier.
3. The weight of both halves of the main span therefore acted on

Fig 7. Damage to the bridge, as reported by SSFM after the collapse[8,13]. Sketch only, not to scale



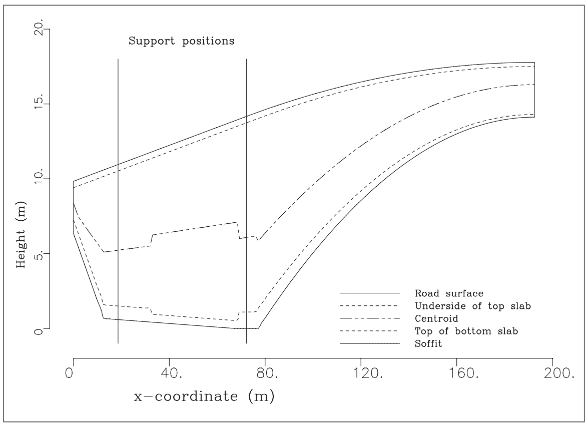


Fig 8. (above) Geometry as-built. Vertical scale exaggerated

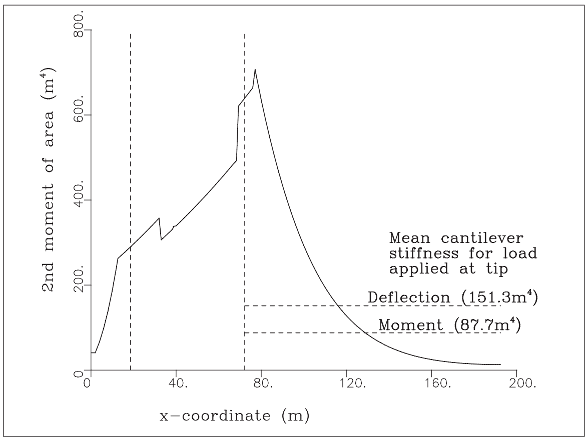


Fig 9. Variation of second moment of area in one half-span

the Koror side. Unable to sustain this increased load, the remainder of the bridge rotated around the Koror-side main pier, shearing the backspan just east of the end pier and lifting it temporarily into the air.

4. The resulting compressive stresses just east of the Koror main pier caused the base of the box girder to crush and displace into the pier itself. The top slab then failed in tension, the backspan fell to the ground and the central span dropped into the channel.

This proposed mechanism is supported by eye-witnesses who heard sounds of popping and concrete falling on metal (presumably concrete spalling from the top slab and landing on the metal services below) for around half an hour beforehand, and saw the Babelthuap side fail first. Further details are inferred from the recorded damage.

The failure mechanism clearly has to be the starting point for any analysis, focussing on the key questions: (1) What could have caused delamination of the concrete in the top deck slab? and (2) Could a shear failure have occurred outright due to high stresses in the box girder webs?

Two key details are apparent. Firstly, relative to the lifetime of the structure (~20 years) the collapse occurred very soon after the remediation works carried out to correct the excessive deflection. Secondly, there was however a time lag between completion of the remediation works (final asphalt laying finished in August) and the actual collapse (26 September).

It seems natural therefore to suggest that the cause of the collapse was directly related to the alterations made to the structure, and involved some kind of time-dependant effect (e.g. creep, shrinkage etc.), which would cause the stresses in the bridge to reach a critical level a month after all work had been completed.

The mechanism of collapse and damage observed on the remains also suggest that the failure was caused by excess shear in the webs or stress (of some kind) in the top deck, just on the 'water side' of the Babelthuap main pier.

Original design

The detailed geometry[11] of one half of the structure is shown in Fig 8, and the corresponding second moment of area is shown in Fig 9. The x-coordinate used here is measured from the extreme back of the bridge, with the rear support at x = 18.6m and the main support at 72.24m. This leaves a cantilever of 120.4m. The same coordinate system applies in both halves of the bridge but is, of course, handed.

There is significant change in stiffness along the length of the bridge, and variation in centroid location, which can have a significant local effect due to shear lag as the stresses redistribute themselves on either side of a discontinuity. Step changes only occur at the support positions and in the back span, and these locations were not implicated in the failure.

The bridge can be analysed using the quoted dimensions and assuming that the concrete is uncracked. The assumption that the structure remained sensibly symmetrical until failure will be discussed in more detail later. When the cantilevers are subjected to a point load at the tip, 87% of the deflection under the load is due to bending in the cantilever, and is equivalent to that in a uniform cantilever with a 2nd moment of area of $151.3m^4$; the remaining 13% of the deflection comes from flexure in the back span. By comparison, when loaded by a moment at the tip, 94% of the deflection at the tip comes from the cantilever, equivalent to a uniform I of $87.7m^4$, reflecting the larger contribution to the moment flexibility that comes from the relatively thin section towards the tip.

If the concrete had a short-term Young's modulus (E) when cast of $30kN/mm^2$ (which may be a little high), the tip deflection due to the beam's own self-weight would have been 0.95m. By assuming a creep factor φ, so that the effective modulus becomes $E/(1 + \varphi)$, the observed creep deflection at the tip of 1.2m would have been produced by $\varphi = 1.26$; this is based on creep affecting the concrete alone and does not take into account the untensioned reinforcement and the reduction of prestress due to creep. These values seem reasonable (although E should perhaps have been lower and φ higher), and the observed deflection due to creep should have been predictable. Fig 10(a) shows the profile of the top chord both as-built, and after 19 years of creep. The vertical scale is exaggerated but the kink that would occur at the centre is clearly visible. The top line shows the assumed profile of the bridge with the dead load removed; it is assumed that the bridge was built so that when complete it reached the desired alignment. The virtual reality image in Fig 10(b) shows, to scale, the kink at mid-span.

Original repair strategy

The original repair strategy seems to have been based on applying additional prestress to the structure over two transverse

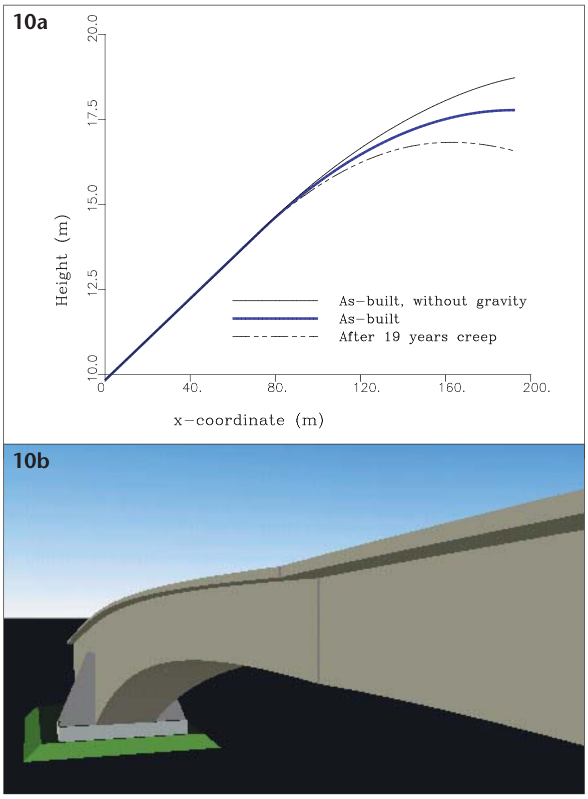


Fig 10. (a) Top chord profile of bridge (b) VR model at mid-span, showing effects of creep

beams near the centre, while keeping the two cantilevers independent. The effect of deflecting the tendons over the beams would have been to induce a net moment at the tip, thus lifting the tips of the cantilevers.

A rough estimate of the effect of applying a moment at the centre can be found by noting that the tip deflection δ will be:

$$\delta = \frac{ML^2}{0.94 \cdot 2\, EI_{\text{eff}}}$$

where the 0.94 factor allows for the flexibility in the back span, L is the length of the cantilever (120.4m) and $I_{\text{eff}} = 87.7\text{m}^4$ as calculated above. If the Young's modulus of the concrete is taken as 18GPa (the value determined by the truck loading), then to remove the unwanted 1.2m deflection a moment of 245MNm would have been needed.

The moment that can be applied in this way is the product of the additional prestressing force and the change in eccentricity that can be induced. The additional prestressing force cannot be made too high for fear of overstressing the section, but a value of about 20% of the initial prestress of 180MN is reasonable since this would replace the prestress losses since construction. The maximum change in eccentricity is limited by the depth of the internal space in the box, which is about 2m. Thus, the maximum possible moment that could have been applied to the tip of the cantilever was about $0.2 \times 180 \times 2 = 72$MNm, or about 30% of that required to eliminate the deflection.

This repair strategy had the disadvantage that four sets of anchorages would be required. In particular, anchorages would be needed near mid-span. Transverse beams would have been required, which would have had to resist the full additional prestress, as would the beams' connections to the existing bridge. It appears that the decision was taken to eliminate these anchorages by carrying the cables across the central gap; the transverse beams would thus only have to be designed for the much lower loads caused by deviating the tendons and eliminating all the complex reinforcement associated with the anchorages.

The disadvantage of the change in procedure was that the structure became statically indeterminate.

Revised repair strategy

The original structure had been built as a pair of separate structures, each founded primarily on one pier that had large inclined piles drilled into the underlying rock: these piers would have had virtually no flexibility in the longitudinal direction. When the additional prestress was applied it would only make contact with the structure at the four deflector beams, with the axial effects passing from one back-span to the other. Freyssinet's dictum that '*the structure must be free to shorten under the action of the prestress*'[14] could not apply, so the cables would cause very little change in the axial prestress in the cantilevers. Worse, the prestress would cause horizontal forces to be applied to the main piers for which they had not been designed.

Thus, it seems that the decision was taken to apply a second additional prestress in the form of flat jacks between the top flanges at mid-span; these applied a force approximately equal to the force in the additional cables. The net result would thus be no additional horizontal force on the main piers.

There thus appear to be three factors that may have caused problems to the structure.

1. The structure had been made continuous. In a linearly elastic material this would have caused no change in the internal moments, which would have remained locked-in as originally built. However, with a visco-elastic material like concrete, continuing deformation would change the bending moments towards those that would have existed had the bridge been built originally as a continuous structure. Most of the concrete was 20 years old, so creep could be expected to be slow, but it would still happen. The effect would be to change the distribution of support reactions, which could in turn affect the shear forces in the beam.
2. The additional prestressing cables would cause vertical loads but very little longitudinal prestress. Could this combination have caused forces at the critical section where failure occurred?
3. Prestress forces induced by jacking apart two structures are very susceptible to losses caused by creep. The axial deformation is small (unlike the initial extension of prestressing cables), so it does not require much change in length of the structure for the prestress to be lost.

When seeking an explanation for the failure, it should be noted that each of these factors preserves the symmetry of the structure about the centreline, and thus none of them can be expected to generate significant shear forces at the location where the initial failure occurred.

In the next sections an attempt will be made to determine the effects of the various actions on the bridge, concentrating on the point at x = 86m on the Babelthaup side, which is approximately where the failure appears to have started.

Effect of continuity

Fig 11 shows the as-built bending moment diagram due to the bridge's own dead weight for one cantilever, as a solid line. There is, of course, no question about these values since the structure was at that stage still statically determinate. The plotted values include the effect of the ballast in the back-span, since this is a permanent load. The ballast was sufficient to ensure a compressive reaction at the end pier, which nevertheless was provided with a tie-down. The peak moment at the main support is 1596MNm, while at x = 86m the moment is 1165MNm and the shear force 27.9MN.

Making the structure indeterminate allows the possibility of moment redistribution due to creep. All other things being equal, the structure would creep towards the bending moment diagram that would have resulted if the structure had been built monolithically. Analysis, taking account of the variation in stiffness, shows that the sagging moment at mid-span under these assumptions would be 128MNm; the resulting bending moment diagram is shown dotted in Fig 11. The difference is much less than would be expected in a beam of uniform cross-section but is understandable given the relatively low stiffness of the tips of the cantilevers.

The amount of redistribution that occurs, and the speed with which it happens, is temperature-dependent and may be significant in determining what happened. England[15] developed a thermal creep analysis and showed that the structure creeps towards a steady state that depends on the temperature of the top and bottom flanges of the bridge. This steady state is not the same as the monolithic moment because the top flange is normally warmer than the bottom, so creep occurs there more rapidly. Xu and Burgoyne[16] used England's analysis to show that the rate of creep depends very heavily on the age of the concrete, and is significantly slower if one part of the structure is older than the rest, which is the situation here.

Xu[17] has carried out an analysis of Palau on the assumption that the structure is continuous, that the top and bottom flanges are at uniform temperatures of 29.2°C and 24.2°C respectively (it is in the tropics!) and that the *in situ* joint is 1m wide. Fig 12 shows the change in support reaction with time, which is a direct reflec-

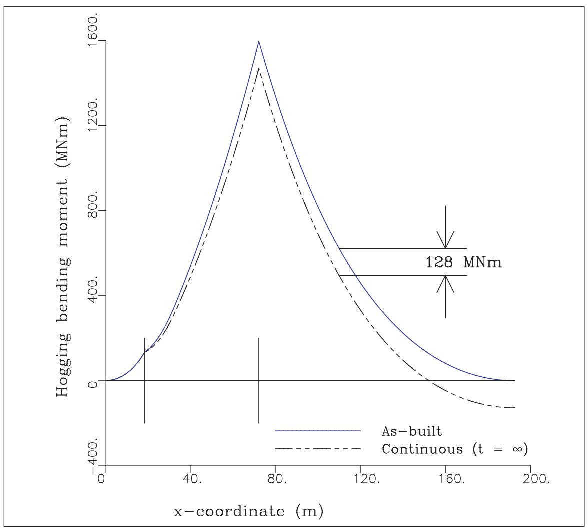


Fig 11. Dead-weight bending moment diagram, including ballast

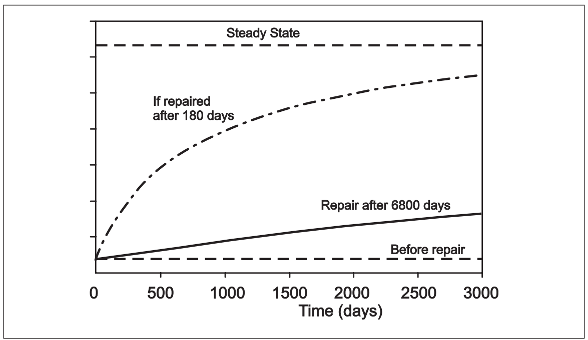


Fig 12. Change in support reaction with time due to thermal creep

tion on the speed with which the redistribution occurs. Detailed values are not given since a slightly different set of assumptions was adopted from that used in the rest of this paper, and what is important is not the *amount* of redistribution that occurs, but the *speed* with which it take place. The lower dashed line shows the moment when the central hinge is made continuous and the prestress has been applied. The upper dashed line shows the steady state value predicted by England's theory, to which the structure would eventually become asymptotic.

The chain-dashed line shows what would have happened if the structure had been made continuous only 6 months after completion. Even then, the relatively old concrete in the main cantilevers would have slowed-down the creep effects caused by making the structure continuous, and after 3000 days (over 8 years), the change in moment has still only moved 80% of the way to its ultimate value.

The solid line shows the situation to be expected when the structure was made continuous after nearly 19 years, as actually happened. The structure does still creep towards the same asymptote, but much more slowly. The change in the 90 days between repair and collapse is only about 1% of the full amount and can thus be sensibly ignored. It can be concluded that there was insufficient time between repair and failure for the creep to have had any significant effect on the overall distribution of bending moments.

Even if these creep effects had occurred, it is doubtful that they would have caused any problems. The effect of the bending moments (from Fig 11) on the top fibre stresses is shown in Fig 13; no account is taken here of any prestress. The as-built stresses are tensile (+ve) everywhere. The reduced inertia at the tip of the cantilever means that the sagging continuity moment has a relatively large effect on the stress near the centreline. The values shown in the figure are for the top fibre, and are thus compressive at the centreline, but there are corresponding tensile stresses in the bottom flange. As far as is known, no continuity steel was placed across the joint when the structure was made continuous, and the additional prestress that was added (to be discussed below), was placed in the top flange.

Thus, it must be expected that these stresses, even if they did get induced in the bridge by creep, would have caused the new continuity joint to open up at the bottom because there was nothing to resist the tension. This would have restored the structure to its original configuration of two independent cantilevers.

These results, taken together, show that:

1. the change in bending moment at the failure location is relatively small (128MNm in 1165MNm);
2. the change would occur very slowly and certainly not within the 6 weeks that elapsed after completion of the repair;
3. it is unlikely to have happened anyway since the continuity joint would probably have opened up in sagging bending.

Thus, it is concluded that no change in stress resultants occurred at the failure location (x = 86m) due to making the structure continuous.

Effect of the continuity cables

The continuity cables pass along the full length of the structure, which at this stage has been made continuous. They make contact with the concrete only at the anchorages and at the deflector beams. The forces exerted by the cable on the bridge are shown in Fig 14. Note that there is no force applied to the bridge at the centreline since the tendon does not touch the concrete here.

The horizontal component of the additional prestress is reacted at the main pier, and only affects the main span if that pier has horizontal flexibility. A study has been undertaken using a three-dimensional finite element analysis to determine how much, if any, of the axial component of the new prestress acts on the central portion of the bridge. Because the main piers are firmly anchored using raking piles, the horizontal forces pass almost entirely through the piles and into the ground. Since there is no published data about the length of these raking piles, nor the stiffness of the ground into which they were anchored, various assumptions were made, but in every case no significant axial prestress passed into the bridge.

Thus, it is concluded that the effect of these cables is limited to the bending moments shown in Fig 15, which have been obtained by analysing the bridge under the loads shown in Fig 14, assuming full continuity at the centre. At x = 86m this gives a moment of 44MNm (sagging) and a shear force of 0.7MN. These are small by comparison with the values due to the beam's own dead weight.

Effect of central flat jacks

Flat jacks were installed between the two cantilevers which were jacked apart with a force of 31MN applied at the centre of the top flange, as shown in Fig 16.

Due to the change in cable profile, a prestressing tendon exerts forces on the concrete all along its length, so the moment is a function of the cable eccentricity at any position, but the force applied by a jack directly on the concrete retains its line of action throughout the structure. The difference in height between the centre of the top flange at the tip, and the centroid at x = 86m is 10.23m,

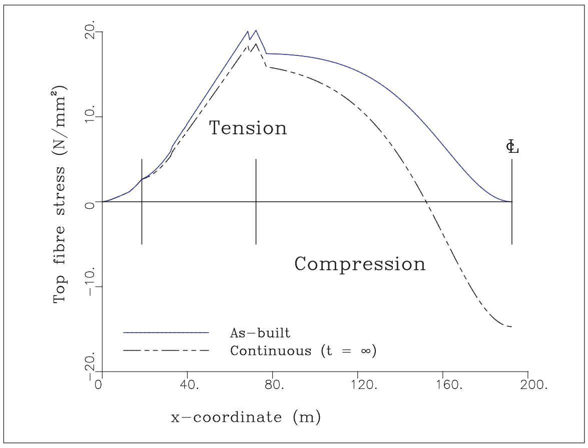


Fig 13. Bending stresses due to dead load and ballast

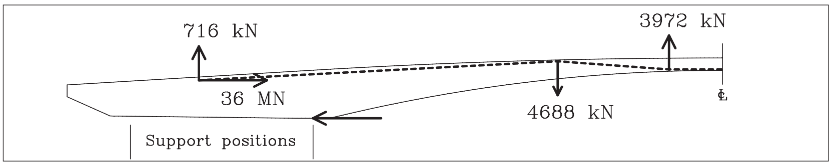


Fig 14. (above) Forces induced by continuity cables

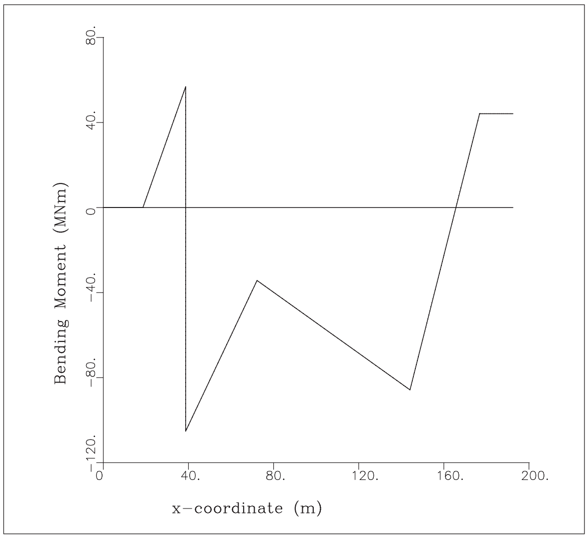
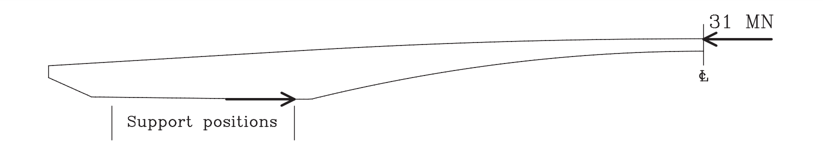


Fig 15. (right) Bending moments induced in the continuous structure due to the continuity cables



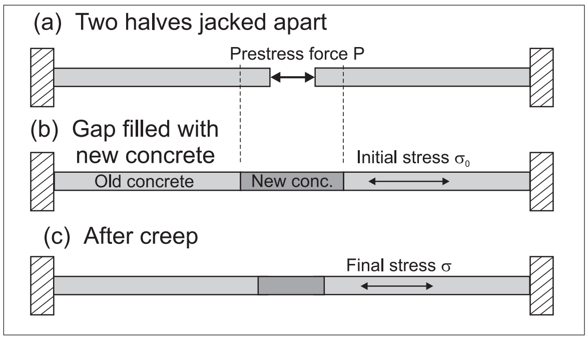


Fig 16. (above) Forces due to flat jacks at centre
Fig 17. (right) Modelling effect of flat jacks over time

so this jack exerted a sagging moment of 317MNm at the critical location. Once the structure has moved, the gap was filled with new concrete, and it is this concrete that continued to apply the force rather than the jacks.

However, the problem with prestress applied by imposed displacement is that it is very susceptible to the effects of creep. In this case, there are two very large amounts of old concrete, which will creep slowly, and a small amount of new concrete, which will creep quickly.

An analysis has been carried out of a simplified problem, as shown in Fig 17, in which two pieces of concrete are forced apart axially and new concrete inserted. This was then analysed using the age-adjusted effective modulus method[18,19], and the variation of force with time plotted as shown in Fig 18.

Several factors were noticed in this analysis. In the first few weeks, which is all that is relevant here, it is the creep in the new concrete that matters; the creep in the old concrete is negligible. Secondly, the size of the original gap left between the two pieces of concrete has very little effect. This observation has two corollaries: (i) it does not matter what assumptions are made about the size of the gap that had to be filled, and (ii) the effect will be proportionally the same at the top and bottom flanges, so both the force and moment induced in the beam will reduce by the same factor.

It is thus concluded that the axial force and moment carried by the new concrete would both have reduced to about 68% of their original values at collapse. This would apply both at the centre and at the failure location. Neither the prestress applied by the flat jacks, nor its loss due to creep would have induced any change in the shear force.

Sudden variations in cross-section

The analysis carried out so far has been based on simple beam theory, which implicitly assumes that the structure has a uniform cross-section; this is clearly not true for this bridge. When the cross-section changes, local stress concentrations can occur since St Venant's principle no longer holds. Nevertheless, if the cross-section changes smoothly, as it does away from the supports, this effect can be expected to be small. To test whether there were any unexpected effects, a three-dimensional finite element model was built using 20-node brick elements. As expected, there are small changes in the stresses from those predicted by simple beam theory, but nothing that would cause concern.

Stress concentration

The initial failure point at x = 86m is about 12 m into the main span, which is about one depth away from the main support and its associated very large reaction force. At this point the bridge is deeper than it is wide, so there are unlikely to be shear lag or effective breadth concerns. St Venant's principle should imply that at this distance from the main support the stresses would be described adequately by simple beam theory. The finite element model has been used to check this prediction; Fig 19 shows a contour plot of the vertical shear stresses in the web around the critical location under gravity loading. The expected stress concentrations at the supports are obvious, but at the failure location there is no sign of anything untoward. The shear stresses are lower than predicted by simple beam theory due to the large compressive force in the steeply-inclined bottom flange, so that the webs do not have to resist the full applied shear force. To these stresses should be added the global effects of the prestress, which can be expected to vary smoothly; the local effects of the main prestress should be limited to the top flange, and should not cause significant additional shear stress at this location.

Thus, it is concluded that there were no peculiar local stress-concentration or shear-lag effects which could have caused problems near the failure zone.

Fig 18. (below) Effect of creep in the *in situ* joint on the stress due to the flat jacks
Fig 19. (bottom) Vertical shear stress in web near failure location, varying from – 3MPa at base to 0.5MPa at top

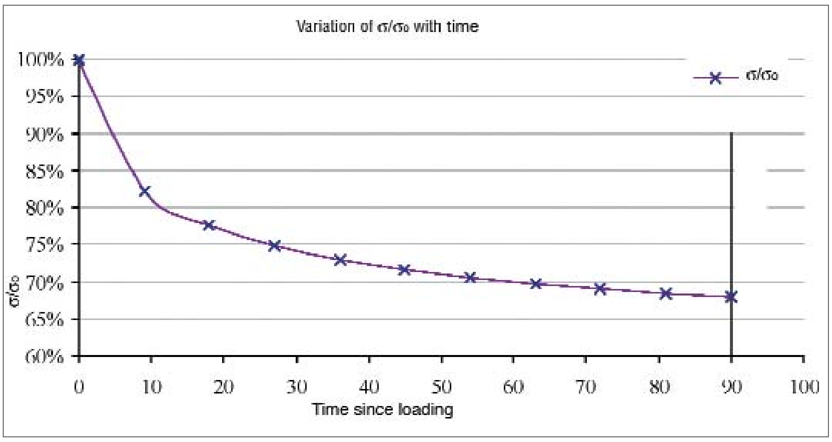
Loss of symmetry

The analyses carried out so far have all assumed that the structure is essentially symmetric about the central hinge. It has already been noted that the tips of the main cantilevers were relatively flexible, which means that a very large movement of one side of the bridge relative to the other would be needed to generate significant asymmetric loading. Such a movement should have been reported, but there is no suggestion of abutment settlement, nor any reported earthquake, which could have caused loss of symmetry. It is thus concluded that the assumption of symmetry is valid.

Summary of analysis

Table 1 shows a summary of the stress resultants at the critical section, as originally built and at the time of collapse. The as-built values follow immediately from equilibrium of the statically determinate structure; the other values are simply a restatement of values obtained above. The notable thing from this table is that it does not show any exceptional values. The magnitudes of the loads induced by the modifications are comparable with the loads originally applied. The biggest effect is that of the flat jacks which, at x = 86m at least, is similar to the loss of the original prestress with time. The corresponding stresses would also show no exceptional values.

Nothing in table 1, nor anything from the detailed finite



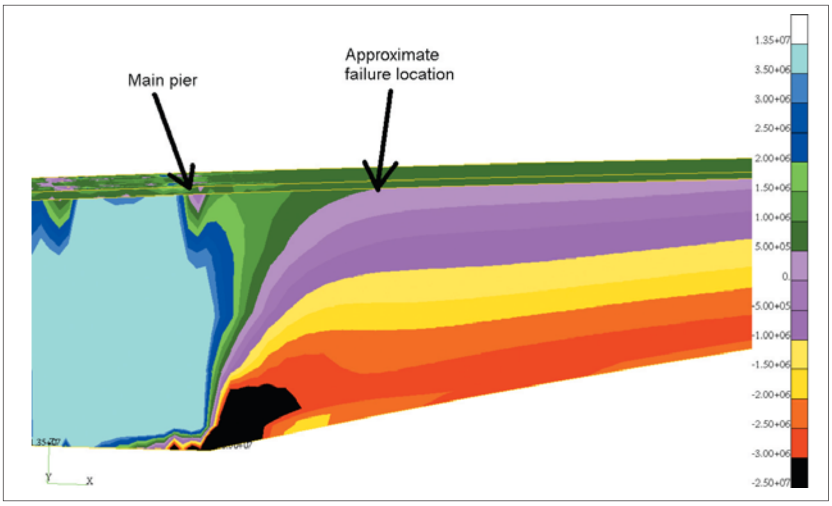


Table 1. Summary of effects at failure location (x = 86m), relative to horizontal and vertical axes

	As-built		6 weeks after repair		
	Due to dead load	Due to prestress (Assuming 25% losses)	Due to creep redistribution	Due to Continuity Cables	Due to Flat Jacks
Moment (Hogging +ve) (MNm)	1165	–885	0	–44	–215
Shear (MN)	27.9	7.3	0	0.7	0
Axial force (MN)	0	121	0	0	21

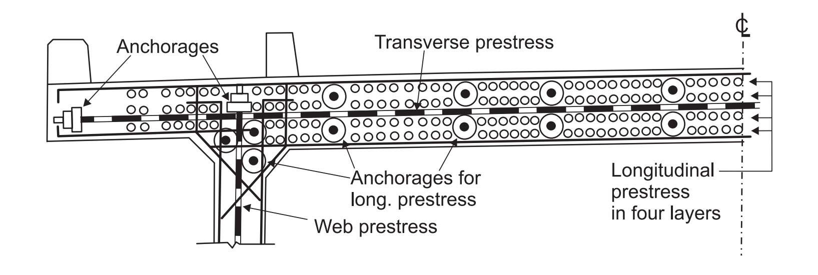


Fig 20. Detail of top flange – web connection[8]

element analysis, gives any hint about the cause of the failure. The list of likely causes, with which this study began, has been exhausted.

Damage to top flange

The one remaining possibility is local damage to the top flange. Reports after the failure[8] indicate that there had been some damage to the top flange, caused by removal of loose material prior to resurfacing. Up to 50mm of material had been removed in some areas. A check of the change in the stresses caused by a uniform removal of this amount of material shows only marginal variation (typically no more than 0.2MPa). It thus seems unlikely that this effect alone could have sufficiently altered the global stresses to cause failure.

There is, however, the possibility that the loss of material caused some local effects. Fig 20 shows details of the top flange (redrawn from ref. 8). There is considerable congestion of the prestressing steel; the top flange, which was 432mm thick, contained up to four layers of 32mm Threadbar tendons running axially and another running transversely. These tendons are available in relatively short lengths so there would have been frequent couplers, and the whole tendon was contained in ducts that would normally have had an external diameter of 51mm but which increased to 76 mm where the bars were coupled[20]. The anchorage for the bar systems also has to be accommodated in the top flange, which required bearing plates 160mm square[20]. The segmental nature of the original construction would have required a large number of both couplers and anchorages.

Fig 20 shows the junction between the top flange and the web to have been particularly congested, with longitudinal, transverse and vertical prestressing tendons crossing one another, as well as anchorages (and presumably couplers) for the various tendons.

Photographs taken during erection[10, 11] do not appear to show bursting reinforcement behind the anchorage positions, nor any through-thickness reinforcement or links tying the top slab together. There was continuous top steel in the transverse direction but discontinuous bottom steel, and relatively little connectivity between the shear steel in the web and the steel in the top flange.

The one paper that reports in detail on the appearance of the bridge after collapse states that the duct spacing reduced to as little as 25mm in the vicinity of the piers[8]. It was also reported that there was no keying in the vertical joints between the different phases of construction, which raises the possibility that all the shear had to be carried by friction across the cast joint. This becomes important if the prestress is lost.

Taken together, the congestion of the prestressing ducts and the lack of tying and bursting reinforcement raises the possibility that local damage to the concrete led to weakening of the unreinforced concrete behind an anchorage. The collapse took place 6 weeks after the resurfacing was complete, which indicates that the process must have been relatively slow. The probability is thus that the damage allowed the stresses locally to rise to such a level that fairly rapid creep of the concrete occurred behind the anchorage, which in turn led to damage to the concrete between the ducts, thus forming a plane of weakness. There would thus have been a sequence of events, which would account for reports that the failure took place slowly. It is probable that these weak planes led to the reported delamination that took place in the half an hour preceding the failure.

It would be possible to analyse the flange-web junction using a detailed three-dimensional finite element analysis, but that requires information about the original design and the as-built nature of the bridge that may not have been available to the investigators and was certainly not available to the authors.

Conclusion

The failure of the Koror-Babelthaup bridge has still not been satisfactorily explained. The investigation reported above shows that the original creep deflection, which led to the repair, should not have been unexpected, although no allowance appears to have been made for it. The repair strategy, using continuity cables and flat jacks, does not appear to have caused major stress changes in the bridge, but neither does it seem to have contributed a great deal to solving the problem of the sag. Thus a secondary strategy was adopted of resurfacing the bridge after packing it with polystyrene blocks to achieve the required profile.

The additional complications, caused by changing the articulation of the bridge from statically determinate cantilevers to a statically indeterminate beam, did not cause the collapse because any effects due to creep would have been too slow to have occurred in time. There was no mechanism to induce sufficient additional shear to have caused a global shear failure.

The conclusion that the failure was probably caused by local damage from over-enthusiastic scabbling of the surface, combined with an insufficiently robust design of the top flange due, in part at least, to congestion of the prestressing steel, rests on the absence of any other credible alternative. As Sherlock Holmes says in *The Beryl Coronet*: 'It is an old maxim of mine that when you have excluded the impossible, whatever remains, however improbable, must be the truth'. If this was the real cause of the problem, then there are lessons for the wider engineering community about the detailing of prestressed concrete slabs. If a major concrete structure can be brought down by one over-enthusiastic worker with a road breaker, then more extensive robustness requirements are needed. It also implies that the structure had been at risk throughout its life.

The authors recognise that they were not in possession of the full facts, and that the conclusions reached here may well be incorrect or at least simplistic. While there may be valid commercial reasons for suppressing the conclusions of the official investigation, major structures should not fail without the lessons being disseminated.

No attempt has been made here to apportion blame to any individual or organisation; if the state of knowledge at the time was insufficient, or the importance of one aspect of the design was not sufficiently understood, then that should not preclude publication of results so that correct conclusions can be drawn now.

Fig 21. Japan-Palau Friendship bridge[23] *(photo: © Tony Jones)*

Epilogue

The bridge has subsequently been rebuilt along the same alignment and with virtually the same span (Fig 21). The work was carried out with aid from Japan and the bridge is known as the Japan-Palau Friendship Bridge. The main span is still a prestressed concrete box-girder but is only 7m deep at the main pier and 3.5m deep at the centre; it is now supported by stay cables[21].

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