Proceedings of ICE
Civil Engineering 161 November 2008
Pages 28–34 Paper 700038

doi: 10.1680/cien.2008.161.6.28

Keywords
bridges; concrete structures; failures

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Lessons learned from the bridge collapse in Palau

The collapse of a record-breaking 240 m span prestressed-concrete bridge in the Pacific Island nation of Palau occurred without warning in 1996. The parties involved were subject to a confidentiality agreement, so no definitive statement has been made as to the cause of the collapse. This paper reports on a study carried out using information in the public domain. It concludes that a repair carried out six weeks before the collapse was not to blame, but did expose weaknesses in the original design. It is recommended that the construction industry should not shelter behind confidentiality clauses but, like the aircraft industry, publish its mistakes so lessons can be learned.

Koror–Babelthuap Bridge in Palau, an island nation in the western Pacific Ocean, collapsed on 26 September 1996 at around 5.45 pm in benign weather conditions and in the absence of abnormal loads.

The failure received headlines in the engineering press for a short time[1–4] but, perhaps because of its remote location, soon dropped from view. The various parties involved with the bridge became involved in a legal dispute about liability, which was settled out of court in an agreement that has not been made public. Non-disclosure clauses have prevented almost any subsequent informed discussion.

At the time of its construction in 1977, the 240 m span Koror–Babelthuap Bridge held the world-record for prestressed-concrete beam bridges. It carried loads satisfactorily for 19 years, albeit with a serviceability problem caused by excessive creep deflections. The bridge was assessed by two teams of engineers, who deemed it safe, and the creep problem was addressed by engineers with wide experience. However, six weeks after these repairs were completed, the bridge collapsed, killing two people and causing such major problems that the Palauan government declared a state of emergency and called for international assistance.

So, why have construction professionals not asked questions about the event? Is the civil and structural engineering profession's understanding of prestressed concrete fundamentally flawed? Was the failure caused by botched repairs or was the repair strategy itself at fault? Or was the failure caused by some hidden flaw in the original design, which became exposed by the repair?

The authors—with no connection to any of the parties involved, merely an interest in the behaviour of prestressed-concrete bridges—decided to try to determine what happened to the bridge on the basis of the relatively small amount of information in the public domain. A paper published in 2006 studied the various failure mechanisms in some detail.[5] This article provides an overview of the 2006 paper and subsequent discussion,[6] and then focuses on the ethical issues associated with bridge collapse.

History prior to collapse

The original 1977 bridge (Fig. 1) provided a link between the two major islands of Palau—Koror and Babelthuap. The latter is the site of the country's international airport and the source of most fresh water; however, approximately 70% of the 21 000 population live

Fig. 1. The 240 m span Koror–Babelthuap Bridge in Palau held the world record for prestressed concrete beam bridges when completed in 1977. However, by the 1990s, creep sag at mid-span had reached a clearly visible 1·2 m

on Koror, which, until 2006, was the site of the capital city. The channel between the two islands is about 30 m deep with tidal flows of up to 3 m/s and steep banks; hence a single-span bridge was chosen.[7, 8]

The original design was symmetric. Each side consisted of a 'main pier' on the channel edge, from which cantilevers extended over the water and met in the centre. Outside the main piers were 54 m approach spans, which rested on the 'end piers'. The main pier was supported on inclined piles that resisted horizontal forces, while the end piers had only vertical piles. The cantilevers themselves, which were segmental and cast in-place, were joined by a slotted connection containing bearings to allow longitudinal movement and rotation of the half-spans relative to one another. The joint ensured displacement compatibility across the span. Fig. 2 shows an elevation of the original design. A box cross-section was used throughout, with fixed widths but varying depth, as shown in Fig. 3.

Each half of the bridge had been built as balanced but asymmetric cantilevers working away

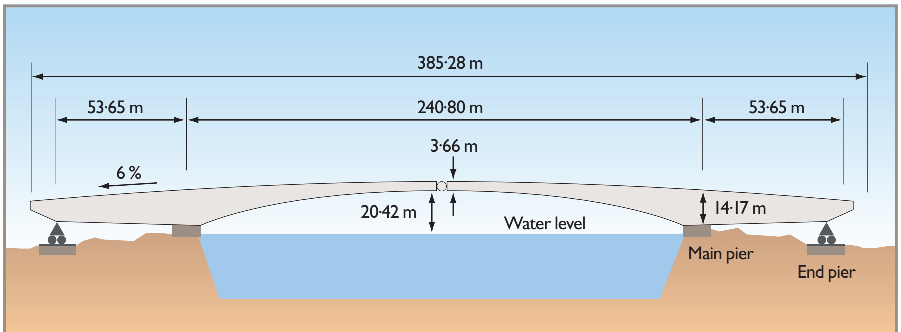

Fig. 2. Elevation showing as-built geometry—the bridge consisted of two balanced, asymmetric cantilevers with a central hinge

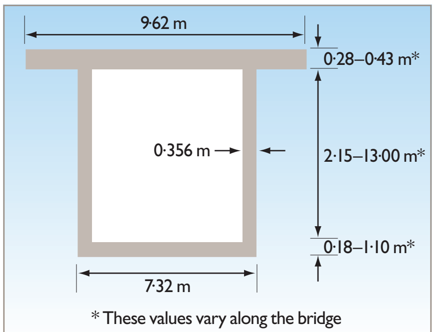

Fig. 3. Simplified cross-section of the cast in situ, prestressed concrete box beam

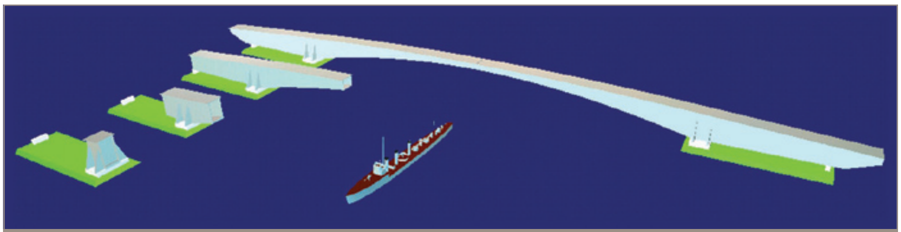

Fig. 4. Graphic showing construction sequence—the back-spans were filled with ballast prior to completion of the cantilevers

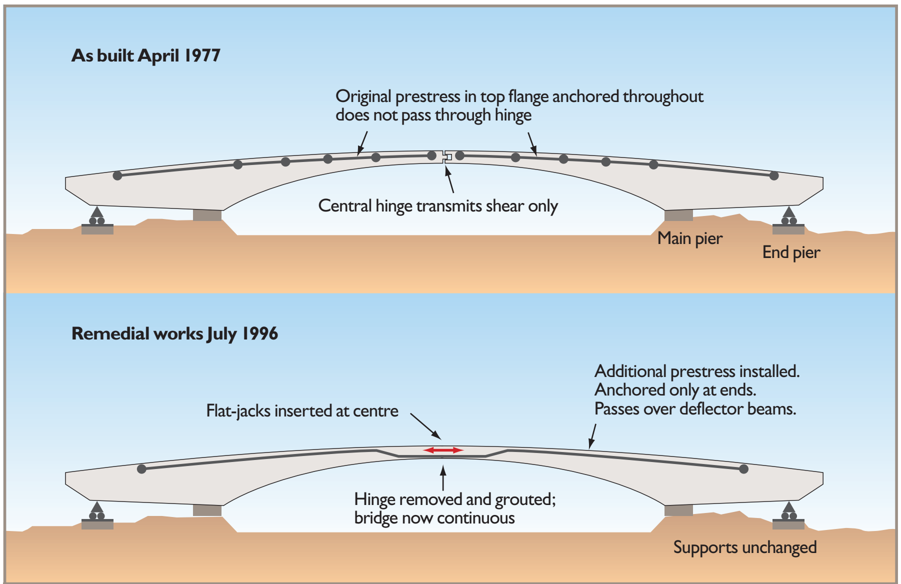

Fig. 5. Elevations showing as-built prestressing in the two halves and additional end-to-end tendons added during repairs 19 years later, when the bridge was made continuous

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. The end supports were provided with a tie-down facility but should at all times have been in compression.

The bridge was in hogging bending throughout and was prestressed longitudinally using 32 mm diameter Dywidag threadbars. There were up to four layers of such bars in the top flange, which also included transverse prestress. There was also vertical prestress in the webs.

After completion, the bridge remained unchanged for 18 years. Over this period, the cantilevers deflected due to creep, shrinkage and prestress loss. By 1990 the sag of the centreline had reached 1·2 m, affecting the appearance of the bridge and 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. Both teams concluded that the bridge was safe and would remain so, but that deflection could be expected to increase by another 0·9 m over the next 100 years. There were concerns that the tips of the two cantilevers could soon come into contact, thus inducing additional and uncontrolled stresses. As a result, the decision was made to tender remediation works to correct some of the sag and prevent further deflection.[9]

A repair proposed by VSL International[10] was accepted, with the work being carried out by a local firm. The four elements to this 'retrofit' were as follows (see 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. These additional tendons were continuous through the bridge, being anchored between the piers on each side. 36 MN of force was applied to these cables, creating a hogging central moment intended to remove 0·3 m of the deflection.
- Insertion of flat-jacks between the top slabs (in place of the central hinge), which were used to apply an additional 31 MN of longitudinal compressive force. These were grouted in place, making the span continuous. The combined effect of the external cables and flat-jacks is 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, thus halving 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 structural remedial works were completed in July 1996 and the surface replacement finished in mid-August.

How and why it collapsed

The bridge collapsed six weeks later (Fig. 6). A report prepared for the US Army[11] 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 water level. A summary of the damaged regions is shown in Fig. 7. The 'pockmarks' indicated in the figure were not mentioned in the report, but were discovered during later failure analysis.

The report (which seems to have been accepted by all parties involved) describes the most probable sequence of collapse as follows.

- 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'.
- 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.
- The weight of both halves of the main span therefore acted on the Koror side. Unable to sustain this increased load, the remainder of the bridge rotated around the Koror-side main pier, shearing the

Fig. 6. Six weeks after repairs were completed, the bridge collapsed into the 30 m deep channel at 5·45 pm on 26 September 1996, killing two people and triggering a national emergency

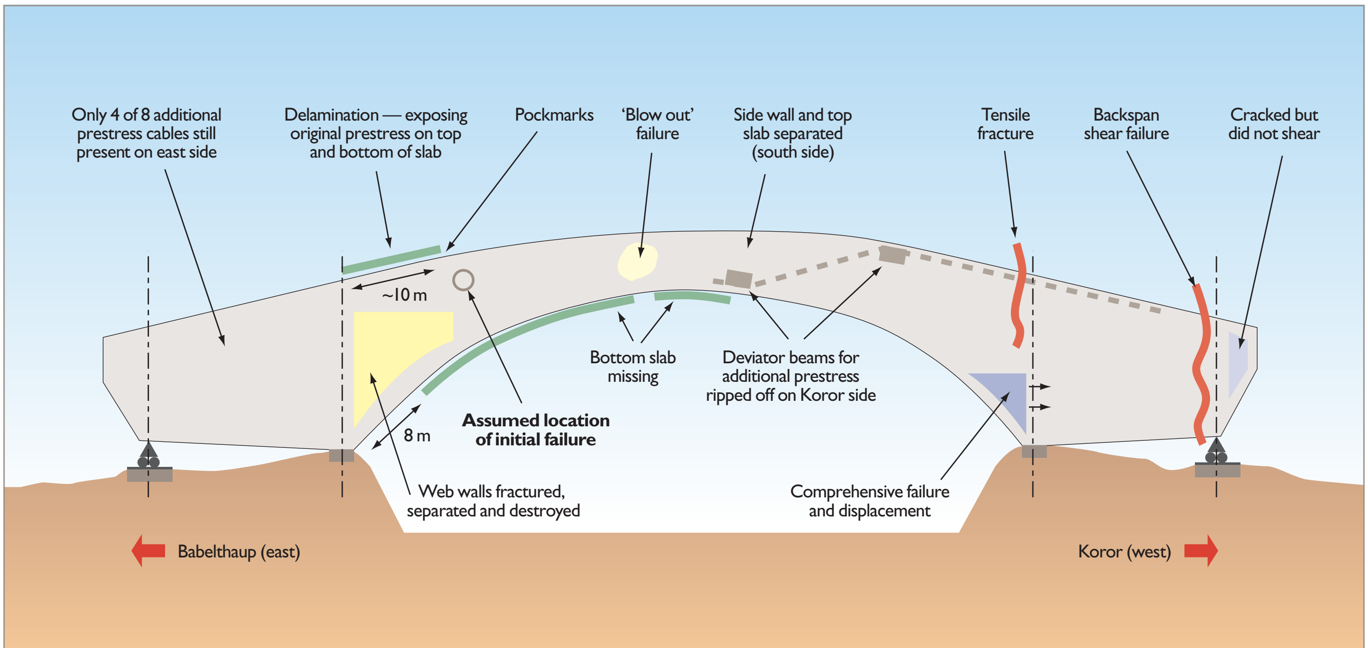

Fig. 7. Summary of damage based primarily on a report for the US Army in 1996 (bridge not to scale)[11]

backspan just east of the end pier and lifting it temporarily into the air.

- 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.

At this point the published record ceases. The various parties became involved in a legal dispute that took place behind closed doors. A settlement was reached, the terms of which have not been made public, but which is known to have included a confidentiality agreement. It is not known whether this clause is subject to any time limit. At one time, the only available references were to websites of various lawyers, who claimed to have reached a satisfactory outcome for their clients. The result is that no conclusive statement has been made by anyone in a position to know the full facts.

Relative to the lifetime of the structure, the collapse occurred soon after the remediation works carried out to correct the excessive deflection, but there was a time lag between the final asphalt laying in August and the actual collapse on 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-dependent effect (e.g. creep or shrinkage), which would cause the stresses in the bridge to become critical a month after all work had been completed.

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

One of the original designers suggested, at a conference in Japan,[12] that the modifications had significantly increased the compressive stresses in the top flange, which thus buckled. Another group, who had inspected the bridge after failure, concluded that the original construction was at fault.[9]

Taking a fresh look

The present authors decided to review all the published material and to re-analyse the bridge from first principles. As full details of that study are given elsewhere,[5] only an overview is presented here. During the original study, attempts were made to contact all the parties involved but most did not reply and none would say anything 'on the record'. It was hoped that publication would prompt comments from those with more detailed knowledge but, apart from some brief discussion by Pritchard and Rush, who had inspected the failed structure on behalf of an insurance company,[6] little has been forthcoming. What follows therefore must be treated with caution because it is based on information that may be wrong and is certainly incomplete.

The study looked at the following structural effects due to the repair strategy.

- Moment redistribution as a result of making the structure continuous.
- Longitudinal forces in the concrete due to the application, and significant subsequent loss, of prestress from additional tendons.
- Loss of additional central jacking forces due to creep.
- Stresses resulting from additional road surface, including void-formers at the central stitch.
- Possible increases in shear, at the failure location, from the all of the above (and other) features of the repair works.

The detailed analysis showed that these effects would have caused some changes to both longitudinal and shear stresses at the critical location, but none significant enough to have initiated the collapse as described. It was thus concluded that the loads induced by the repair strategy would not, by themselves, have been sufficient to cause failure. But it is too much of a coincidence to believe that the repair process did not in some way have a bearing on the collapse.

Attention therefore turned to the detailing of the original prestress and comments on the observation of 'pockmarks' in the top slab. These were believed to have been caused by over-enthusiastic use of a road breaker when removing the wearing course of the original structure. Other reports imply that the original non-structural lean-mix surfacing was removed easily over most of the deck surface, but was firmly bonded to the structural concrete in precisely the area where the failure

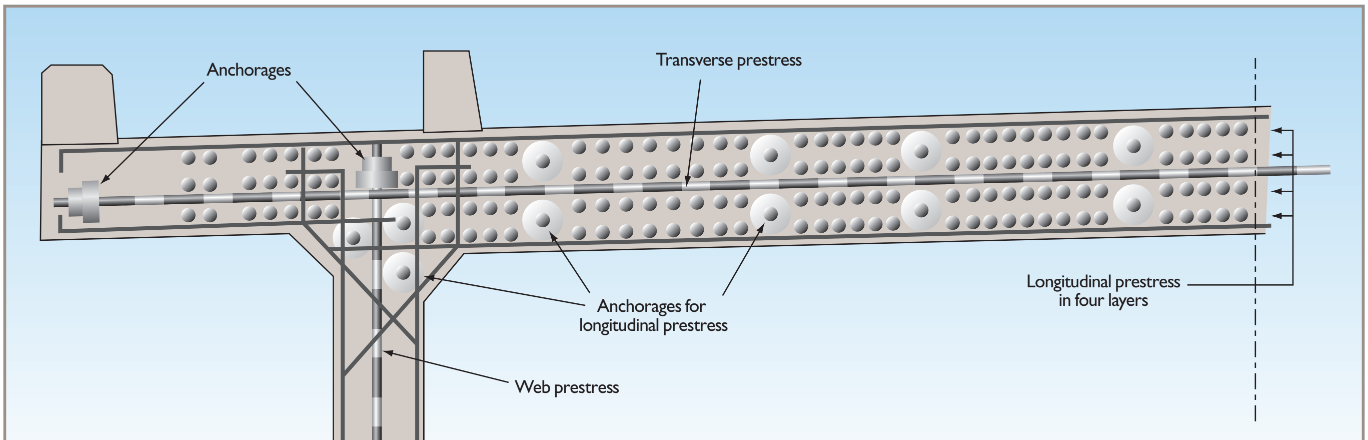

Fig. 8. Cross-section of as-built top flange near where the bridge initially failed—un-reinforced concrete around some of the upper prestress anchorages may have been damaged during replacement of the road surface six weeks earlier

eventually occurred.

The amount of material removed is unlikely to have caused global effects, but it is possible it caused some local effects. Fig. 8 shows details of the top flange[9] and Fig. 9 shows a photograph of this region from inside the box, taken after the collapse.[6] What can be seen from these figures is heavy congestion of prestressing ducts, anchors and couplers. Reports from observation after the collapse[9] indicate that the prestress spacing reduced to as little as 25 mm in the vicinity of the piers. In addition, the absence of through-thickness or bursting reinforcement is notable, even in the vicinity of the anchorages, and there seems to be relatively little connectivity between the shear steel in the web and the steel in the top flange.

It was thus postulated that local damage to the concrete led to weakening of the unreinforced concrete next to an anchorage. The collapse took place six weeks after resurfacing was complete, which indicates that the process must have been relatively slow. The probability is that the damage caused by breaking out the old surfacing allowed local stresses to rise to such a level that fairly rapid creep of the concrete occurred next to an anchorage. This in turn led to damage to the concrete between the prestressing layers, thus forming a plane of weakness. These weak planes led to the reported delamination that took place in the 30 minutes preceding the failure. There would thus have been a sequence of events accounting for reports that the failure took place slowly.

The scenario described above is improbable but plausible, and is what remains when more likely causes of failure have been eliminated. It is highly speculative, and questions remain about who should be blamed and what lessons should be learned. These questions go beyond large-span prestressed concrete and apply to many structures.

Design and construction

The original construction method does not seem unreasonable given the location. By building the main span as a pair of cantilevers, most activity can take place at the leading edge using travelling falsework.

Balanced-cantilever construction uses exactly the same logic today. In normal balanced-cantilever construction, the structure is made continuous by an in situ joint and continuity cables. The articulation of the bridge has to be altered to cope with axial shortening of the bridge due to the effects of prestress, creep and thermal movement. Most balanced-cantilever bridges are built on tall piers, which can flex or be articulated, and axial movement taken to an expansion joint at one end. That would have been difficult at Palau since one of the main piers would have needed some form of release to allow horizontal movement, which would have been difficult to accommodate because of the large forces involved. It would have also introduced a significant future maintenance headache.

The congestion of the prestressing bars, couplers and anchorages in the top slab is

Questions remain about who should be blamed and what lessons should be learned. These questions go beyond large-span prestressed concrete and apply to many structures

Fig. 9. Underside of the top flange in the initial failure area showing delamination of the lowest layer of closely spaced prestress ducts and lack of vertical links[6]

noteworthy and must have caused problems during construction. Ensuring compaction of the concrete between and beneath the prestress bars must have been difficult. It would be interesting to see the justification for the lack of secondary steel in the top slab, especially in and around the anchorage locations where local stress concentrations would be expected.

The failure to allow for long-term creep in the original structure is surprising. Most engineers have come across situations where concrete creep has had some effects, if only in the unexpected deflection of a reinforced concrete lintel. Most such effects are small and at worst cosmetic, and failure to allow for them is forgivable. But in some cases, where deflection is critical, creep must be treated properly—Palau should have been such a case. A 1% deflection seems small but, on a 120 m cantilever, it is significant.

The design of a record-breaking bridge such as this should have been assigned to engineers who could look beyond simplistic code provisions. Creep must have occurred during construction, and must have been taken into account to achieve the desired profile.

It has been reported[6] that the original contract was awarded for a low price to a contractor that subsequently disbanded. During construction, the original project engineer was dismissed after complaining about concrete quality control, which was probably justified given the later measurement of the stiffness of the concrete, which showed a very low value.

Comments on repair

The apparent bridge failure mechanisms do not seem to implicate the repairer, but questions should be asked about the assessment of concrete quality. Measurement of bridge stiffness should have raised concerns about the existing concrete quality. Should cores have been taken? In such a congested top flange, would this have been feasible anyway?

The stress changes predicted by the modifications were fairly small and have caused the stresses to significantly exceed those seen by the structure before losses in the original prestress.

It is also valid to ask whether it was wise to change the articulation of the bridge. The original structure was statically determinate. Fixing the central hinge, which was done on cost grounds, introduced the possibility of an uncertain load distribution.

Conclusions and lessons learned

The final conclusion of the present study is that the failure was not caused by the repair, but that unexpected flaws in the original design and construction were exposed by the repair. It is reasonable to ask what lessons should be learned.

The bridge appears to have failed because a poor design was badly executed. Does the fault lie with the original designer, who produced a very congested design, or with the contractor, who did not seem to have been able to produce concrete of adequate quality? Should the client take some responsibility—*caveat emptor* (let the buyer beware)? If a job is tendered at less than the price expected, should the client accept gratefully or ask why? The old adages apply: ‘you get what you pay for’ and ‘the contract goes to the tenderer that has made the biggest mistake.’

What lessons should the structural engineering profession take from Palau? There seems to have been a failure to allow for buildability and a failure to allow for long-term creep effects in a structure where they would have been important. Should a client allow a design that pushes the boundaries to be carried out by designers who simply apply existing code provisions or by non-specialist contractors?

Is any interest served by confidentiality agreements that prevent lessons being learned from construction mistakes? The blame/litigation culture in the civil engineering industry is a serious impediment to learning from the mistakes that occur from time to time. The contrast with the aircraft industry is marked. It can be equally litigious, but there is compulsory reporting of mistakes and widespread sharing of technical information. Aircraft builders, airlines and airports cannot shelter behind confidentiality clauses; information must be shared so that everyone can benefit.

The work of bodies such as the UK Standing Committee on Structural Safety (Scoss) and its associated scheme for confidential reporting on structural safety (Cross) mean that, in a UK context at least, it should be possible for lessons to be drawn from failures or bad practices. But Scoss relies on a committee of volunteers, albeit wise and willing, while Cross relies on well-intentioned engineers making reports that would probably get them sacked if their bosses knew about them.

In an international context there are issues of jurisdiction, but the aviation industry has managed to overcome these problems. The construction industry should do the same.

Acknowledgements

Figures 1 and 6 are from postcards published by the late W. E. Perryclear, Island Photography, Box 1784, Koror, Palau 96940. Fig. 9 is reproduced with permission from Martin Rush, QED Singapore.

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