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Restoration and Widening of the Tasman Bridge

Remise en état et élargissement du pont Tasman

Wiederherstellung und Verbreiterung der Tasman Brücke

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SUMMARY
Three out of 22 river spans of the Tasman Bridge were destroyed by ship collision. The paper describes the method of debris survey in the deep water site conditions and the method of restoration determined therefrom. The unusual engineering tasks involved in the work included temporary repairs, demolition, major steel falsework, unusual vertical piling, the incorporation of a long steel span into the prestressed concrete bridge and erection techniques used. The whole bridge was widened concurrently with the restoration.

RESUME
Trois des 22 portées du Pont Tasman furent détruites par suite d'une collision d'un bateau avec les piles du pont. Le rapport décrit la méthode d'examen des décombres et la méthode qui en résulta pour la remise en état. Les tâches inhabituelles que l'ingénieur rencontra ici comprenaient: des réparations temporaires, démolition, des échafaudages importants en acier, la mise en place de pieux verticaux, l'incorporation d'une longue portée en acier dans le pont existant en béton précontraint ainsi que les techniques de montage utilisées. A l'occasion de cette remise en état, le pont a été élargi dans toute sa longueur.

ZUSAMMENFASSUNG
Von den 22 Flussöffnungen der Tasman Brücke wurden drei durch einen Schiffsstoss zerstört. Das Referat beschreibt die Methoden der Schadensfeststellungen im tiefen Wasser sowie die gewählten Wiederherstellungsmassnahmen. Zu den aussergewöhnlichen Ingenieurarbeiten gehören unter anderem: provisorische Reparaturen, Abbruch, das Erstellen eines Stahlschalgerüsts, ungewöhnliches Bohren von Pfählen etc. Die ganze Brücke wurde gleichzeitig saniert und erweitert.

1. INTRODUCTION

The Tasman Bridge (Fig. 1) crosses the River Derwent and provides the only road link between the eastern and western shores of the City of Hobart which is the capital of the southern Australian State of Tasmania. The bridge was constructed in 1959-64 and design and construction of it was reported by TROLLOPE /1/, BIRKETT /2/ and NEW /3/. It is about 1.5 km in length and when opened in 1964 provided a four lane high level crossing of the river. In 1975, 45,000 vehicles per day were using the bridge and between 1964 and 1975 there had been about 12,000 ship transits.

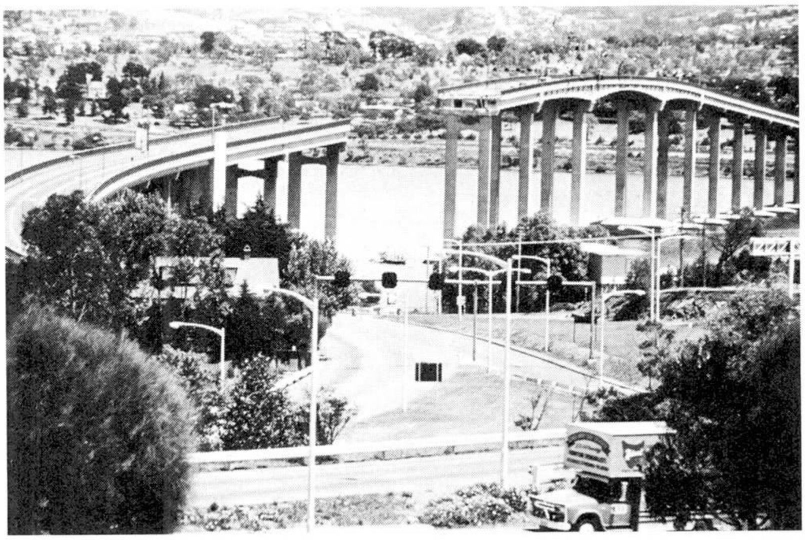

Fig. 1 Tasman Bridge after collapse

At the bridge site the river is wide and deep, water depths generally being in excess of 30 m. The bridge itself consists of three separate structures (Fig. 2), the navigation spans in mid river, 13 x 42.7 m spans in the western main viaduct and 6 x 42.7 m spans in the eastern main viaduct. The original design recognised the possibility of a ship collision although the

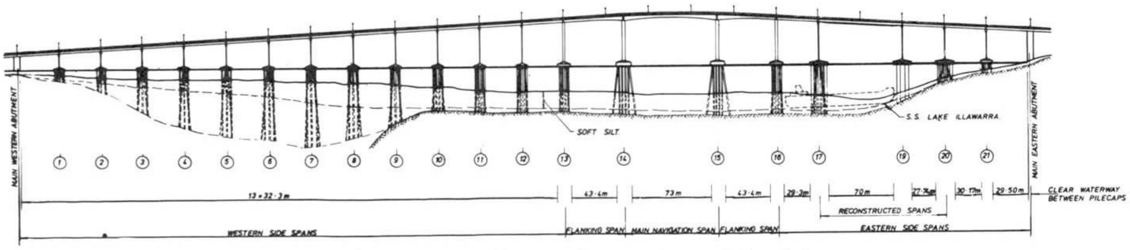

Fig. 2 Elevation of restored bridge

probability of this was thought to be very small. Accordingly the following provisions were made.

- Gravity fender protection to the navigation span piers.
- A special deck continuity detail at the main viaduct piers, so that in the event of a pier being knocked over the adjacent spans would break away cleanly and fall without damaging the remainder of the bridge.
- Stay bolts between the navigation spans and main viaducts, to provide longitudinal support to the latter in the event of the collapse of some viaduct spans due to a ship collision.

In January 1975, the ore carrier S.S. Lake Illawarra, fully laden and displacing 17,000 tonnes, collided with Piers 18 and 19 of the bridge bringing down these piers together with three spans of the eastern main viaduct and leaving a gap of 128 m. The centre one of the three spans fell on the bow of the ship which quickly sank, the tragedy resulting in the loss of twelve lives. The severence of the only road link also caused quite substantial social disruption, as the eastern shore has mainly dormitory suburbs, with the majority of the city's infrastructure being on the western shore.

The consequences of the accident to the remainder of the bridge were relatively minor as it had performed in accordance with design provisions. Span 16-17, the only remaining eastern main viaduct span on the river side of the gap, had remained in position supported by the stay bolts. Pile Cap 20 had suffered severe damage when it was hit by the falling span 19-20, whose trajectory of free fall was intercepted by one end hitting the bow of the ship. The columns on either side of the opening were deflected towards the gap at the top by up to 200 mm and the lower part of the columns was severely cracked. Otherwise the remaining 19 spans of the bridge were undamaged.

Subsequently a Court of Marine Inquiry /4/ determined that the cause of the accident was an error in navigation.

As the majority of the bridge was intact, the decision to renovate rather than rebuild was a straight forward one, as not only would it be much cheaper but also very much faster, the latter being of prime importance under the circumstances.

2. ADMINISTRATIVE AND CONTRACTUAL ARRANGEMENTS

The complicated physical situation following the disaster was compounded equally by a complicated administrative situation with overlapping jurisdiction by many authorities concerned with the bridge, the ship and the port. This situation was overcome and the work expedited by the formation of a separate authority, the Joint Tasman Bridge Restoration Commission, whose responsibility covered both the bridge and the ship. This has been reported on by KNIGHT /5/.

One of the early and most important decisions reached by the Bridge Commission was that an immediate attempt to salvage the ship was not the best course, that priority should be given to the bridge restoration and that the restoration work should be so devised as to avoid substantial removal of major sections of the ship.

The Bridge Commission appointed Maunsell and Partners Pty. Ltd. as its consulting engineers and the State Government appointed a leading Australian contractor, John Holland (Constructions) Pty. Ltd. to be responsible for the bridge work. The basis of Holland's contract was a mixture of dayworks performed by their own plant and site labour, together with project management provided by them for the procurement of materials and elements fabricated off site. Design proceeded in conjunction with construction as time was not available to do it before work commenced. Design liaison between the consulting engineers and contractor was excellent and this was of considerable advantage in ensuring that design detailing matched construction techniques to be employed.

3. DEBRIS SURVEY

Investigations revealed that the bridge could not be restored without additional piling. Due to the 35 m height of the deck, even a single span truss solution to span the 128 m gap would need additional piles to resist transverse wind loads. Thus with the ship close and nearly parallel to the bridge, and a further 7,000 tonnes of concrete debris in the mud at the bottom of the river, the need to do an accurate survey of the debris was of prime importance. There was no question of trying to remove the debris in this depth of water as the large concrete sizes (pile caps 500 tonnes) would have made the work hazardous, difficult and time consuming. Before commencing this survey a funadmental decision was taken that the new piles should be large diameter vertical cylinders and if necessary, rock anchored at the base to reduce their lateral flexibility. This avoided having to do the survey on a "rake" and the imposed verticality significantly improved its accuracy.

The debris survey required extensive use of divers in difficult conditions of very deep water and a visibility at best of about 1 m using torches. Three different methods were used. The principle method was the use of underwater ultrasonic survey equipment developed by the University of Tasmania and reported on by LAWSON /6/. The equipment consisted of a transmitter carried by the diver and placed against the object to be located, an array of four receivers mounted on a semi-submerged platform and a central control unit providing control signals and outputs. By measuring the time taken for the sound wave to be transmitted to the receivers, which had been accurately located by surveying from the shore, and using simple co-ordinate geometry calculated by a computer, a point on an object was quickly located within 100 mm. The other survey methods involved a probe consisting of 6 m of universal column section attached to a long steel cylinder, and diamond drilling through the debris.

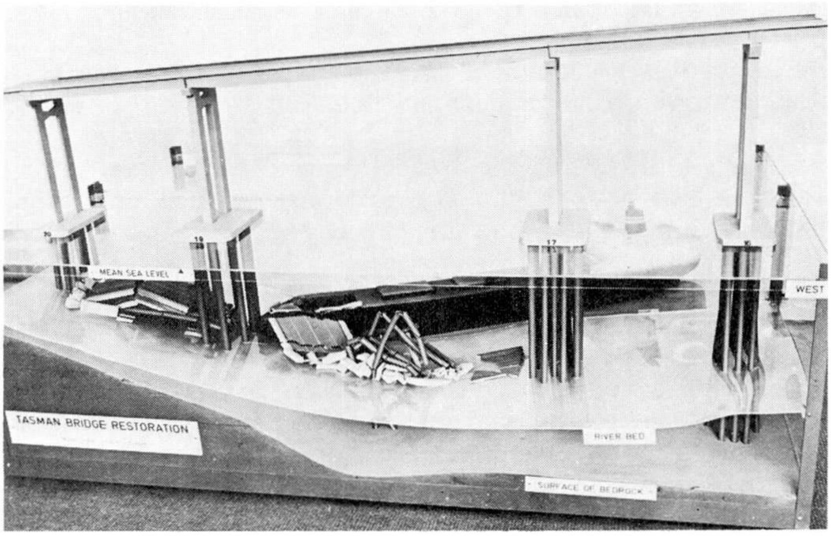

Fig. 3 Debris model

The debris survey took about three months and in the end all the broken bridge pieces were accounted for. A model was built to show the underwater picture (Fig. 3). During the debris survey it is to the credit of the Bridge Commission that pressures to make an early decision on the method of restoration were resisted. Five alternate schemes for reconstruction ranging from a single span scheme to a triple span scheme were examined in some detail. As the underwater scene unfolded the schemes were discarded one by one because debris prevented the placement of new piles, and the selected scheme only became evident at the end of the survey period when a complete picture was available. The only clear space available to establish a new pier was at the site of the original Pier 19 and this involved placement of new vertical piles between the existing broken off raking piles. The decision to delay the selection of a reconstruction method until the survey was completed was vindicated by job performance as no delays occurred due to new piles striking debris.

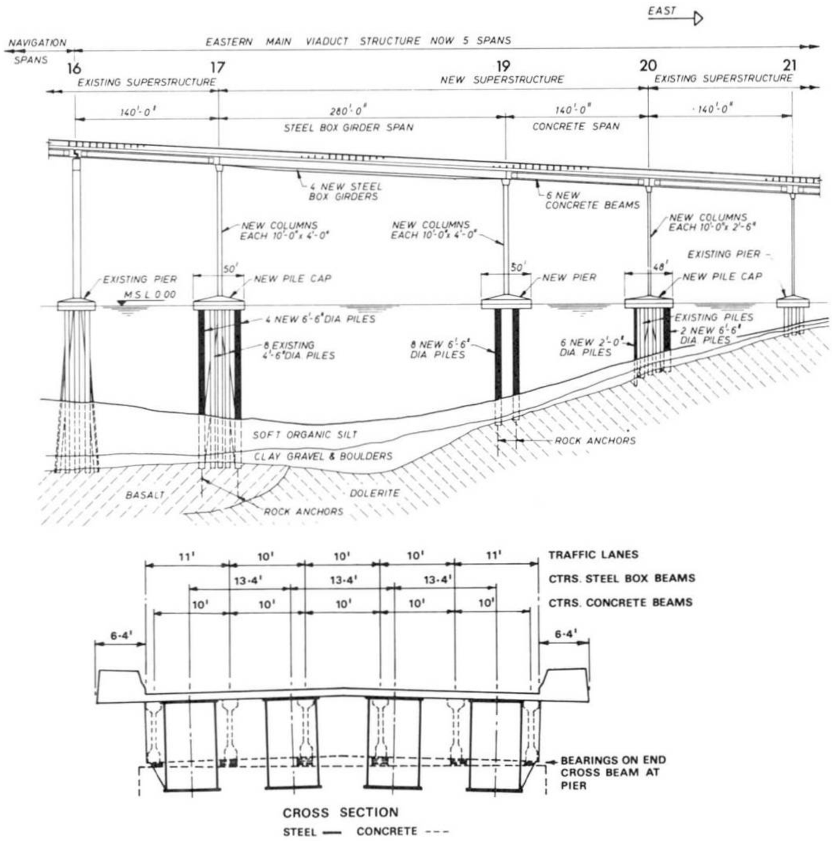

Fig. 4 Details of restoration

4. METHOD OF RESTORATION

The method adopted is shown in Fig. 4 and has been reported on previously by LESLIE /7/ and /8/. As the broken 1.4 m diameter piles at Pier 18 (40 m to bed level being the deepest part of the river), rose 15 m above the mud level this pier was omitted and the method involved the replacement of the three fallen spans by two spans; an 85 m steel span 17-19 and 43 m concrete span 19-20. Cross-sections are shown in Figs. 6 and 7. The scheme involved 14 new 2 m diameter vertical piles rock anchored at the base, some of which were to be used both as falsework piles and permanent piles. Foundation conditions were predetermined using one borehole per pile and were good, consisting of almost fresh dolerite at Piers 19 and 20 and partly weathered basalt at Pier 17. At Piers 17 and 20 the existing columns and pile caps had to be demolished and replaced. At Pier 17 additional piles, working in conjunction with the eight existing 1.4 m diameter raking piles, a bigger cap, and heavier columns were required to resist the greater loads from the double span. At Pier 20 there are no additional loads, but it was considered necessary to replace the badly damaged pile cap and columns, and the falsework piles required to achieve this were incorporated into the permanent structure, although not strictly required from a strength viewpoint. As there was a considerable time saving involved, the replacement of the substructure at these two piers was to be undertaken without demolishing the flanking spans (total mass 1,250 tonnes each) supported by them.

The method of restoration was determined in June 1975 and at that time it was expected that reconstruction would be completed at the end of 1977 after 30 months. At the outset it was hoped that this forecast could be improved upon. The construction relied heavily on the use of water borne equipment supplied by the Contractor. The main items were, Derrick Barge No. 1 displacing 280 tonnes with two triple drum five tonne mooring winches, a 30 tonne Favco stiffleg derrick (32 m jib), Kobe K42 pile hammer and Franki machine; Derrick Barge No. 2 carrying a Favco 1500 tower crane with jib combinations to 48 m and a crane barge with hand mooring winches carrying a 35 tonne Linkbelt Crawler crane with 21 m jib.

5. SECURITY

Before piling work commenced it was necessary to ensure that the damaged existing structure was secure. The work involved the deflected and cracked columns at Piers 17 and 20 and the damaged Pile Cap 20.

For the columns the work consisted of an ultimate load check, incorporating an accurate theoretical assessment of P-Δ effects, to demonstrate that the deflected columns were safe, and arising from this the provision of tie bolts between the deck and crossheads to maintain a pinned connection at the top of the columns. In addition the precautionary measure was taken of welding tie plates across the Pier 16 expansion joint. Some time later when it was possible to examine the Pier 16 stay bolts, some plastic strain and necking was evident and the caution shown earlier proved worthwhile.

The security of Pile Cap 20 was more difficult as it had lost about one third of its volume (Fig. 5) and the remainder was extensively cracked and in some places crumbled. Its load carrying capacity was unknown and although it had to be replaced later it was decided to do temporary repairs before any major work commenced. These were in two parts:-

- Replacement of the concrete which had been removed without attempting to repair or remove and replace the cracked concrete.

- Construction of a temporary steel pile cap above the existing cap (Fig. 6). Its purpose was to enable column loads to by-pass the existing cap and be transmitted directly into the piles. This action was achieved by stressing the steel cap to the columns and by casting concrete plinths on top of the existing cap, directly above the piles, and stressing the steel cap to the piles by means of rock anchors drilled and grouted into them.

These security works contributed significantly to the confidence of the site labour force at the commencement of construction.

6. PILING

Details of the new vertical piling are shown in Fig. 7. 2 m diameter was selected as being the largest size which could be

Fig. 5 Damage to Pile Cap 20

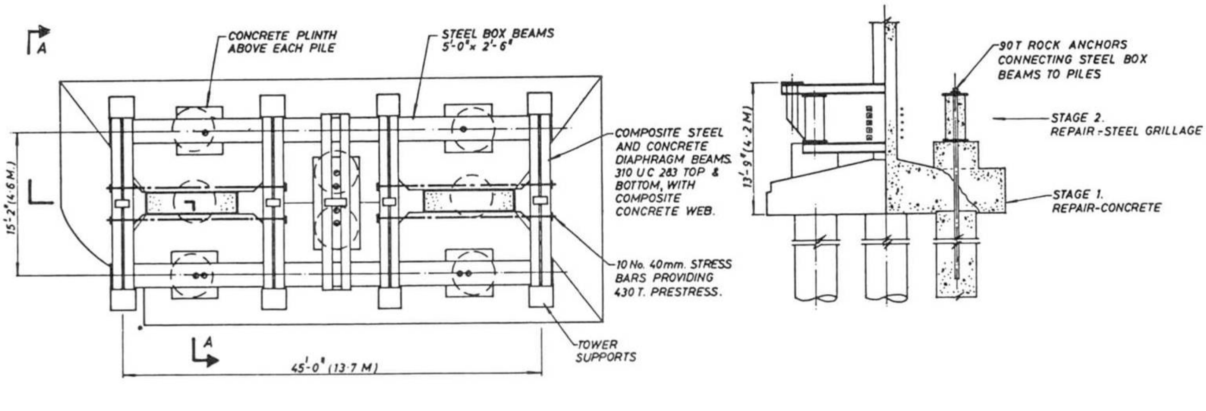

Fig. 6 Pile Cap 20. Temporary steel cap

reasonably handled. Base fixity to the piles was provided by means of rock anchors. At Pier 19 where all piles are vertical deck deflection under a 45 m/sec. transverse wind is 40 mm being 1 in 1,500 of the deck height above foundation level. The maximum loads per pile at this pier were 10,630 kN and 4,650 kNm.

The principle design features of the piles were; casing designed to be dewatered; reinforced concrete tremie plugs (40 MPa) to give F of S against flotation of 1.1; rock anchors designed for maximum corrosion resistance and each one tested in accordance with FIP recommendations. For safety reasons the test load was 70% UTS and lock off load 54% UTS.

During construction advantage was taken of the deep water to tow out full length casings (maximum 50 m long and 60 tonnes mass) and rotate them to the vertical position using flotation principles.

After founding, the tremie plug was poured and rock anchor holes drilled from the top through a casing cluster inserted into the wet tremie concrete.

Back-grouting and redrilling using a rock roller enabled the anchors at Pier 19 and 20 to be placed in the dry using immersion principles.

At Pier 17 due to frequent rock joints the anchor holes could not be sealed and the installation method was modified to enable anchors to be grouted in a flooded pile (Fig. 7). Individual holes were kept grout filled while the grout leaking into adjacent holes was flushed out. A reservoir at the top of the plug enabled the grout quality to be monitored by underwater TV camera while visibility was maintained by downward circulation of fresh water.

The remaining pile operations were straight forward.

7. TEMPORARY WORKS

Major on site temporary steelwork totalling 1,000 tonnes approximately was required. The major items were steel towers at the three piers, arrangements for jacking spans both horizontally and vertically, the underwater support of existing pile groups using 6 m deep steel trusses, and steelwork for temporary diaphragms and articulation.

The steel towers fulfilled many functions, including support of existing spans during column demolition, bracing for new concrete columns during construction, supports for the erection derricks and landing points for the new beams. The construction sequence involved in the use of the tower at Pier 17 is shown in Fig. 8.

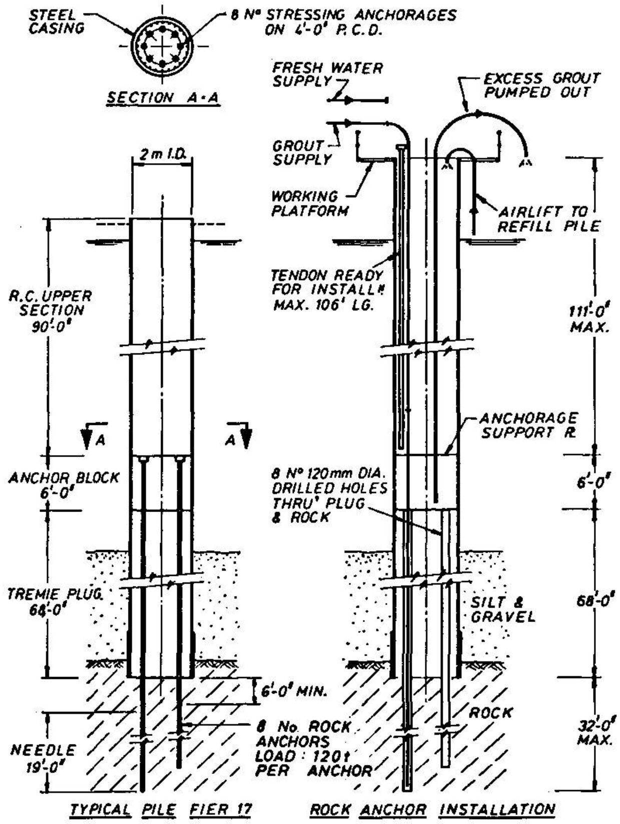

Fig. 7 Pile details

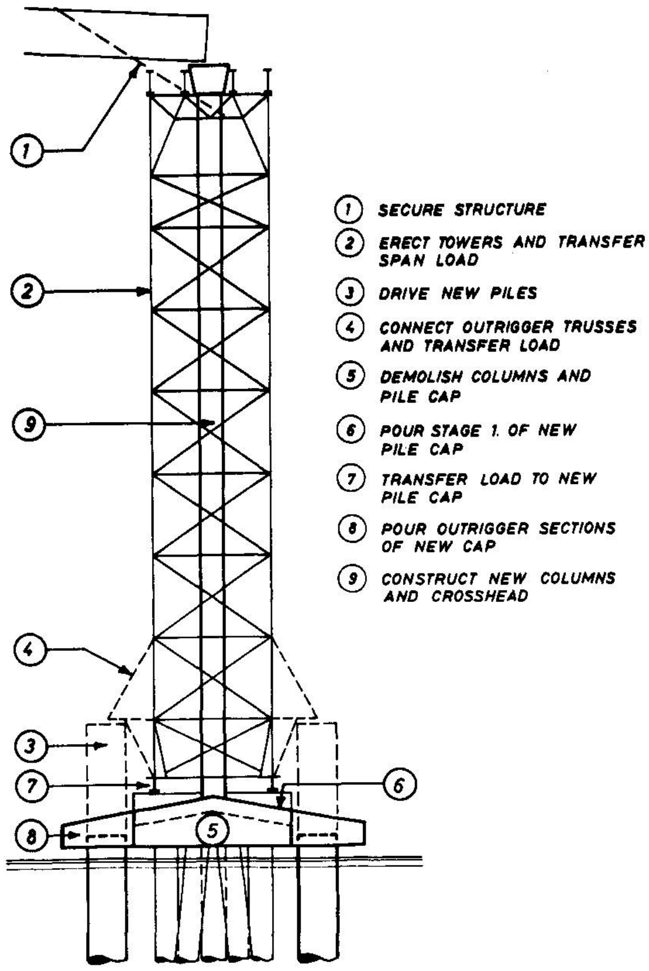

Fig. 8 Pier 17 construction sequence

8. DEMOLITION

Piers 17 and 20 had to be demolished to water level. The original crossheads were heavily reinforced concrete 1.25 m x 1.7 m x 16.5 m weighing 81 tonnes. They were cut into four equal pieces using the thermic lance technique, skidded from under the span across the top of the temporary steel tower and lowered to water level. The process involved burning horizontal holes through the top and bottom reinforcing mats and also through the distribution steel. The section was then split using hydraulic wedge splitters.

The original columns were made up of precast concrete blocks, vertically stressed together with Macalloy bars. They had a cross-section of 3 m x 0.75 m and were cut horizontally using a thermic lance and hydraulic wedge splitters. The 25 tonne pieces were lifted out through the top of the falsework tower (Fig. 9). Twelve working days were required to remove the two legs of the Pier 20 column.

The original pile caps measured 16 m x 7.6 m x 1.8 m and were demolished using track mounted pneumatic equipment working under the falsework towers.

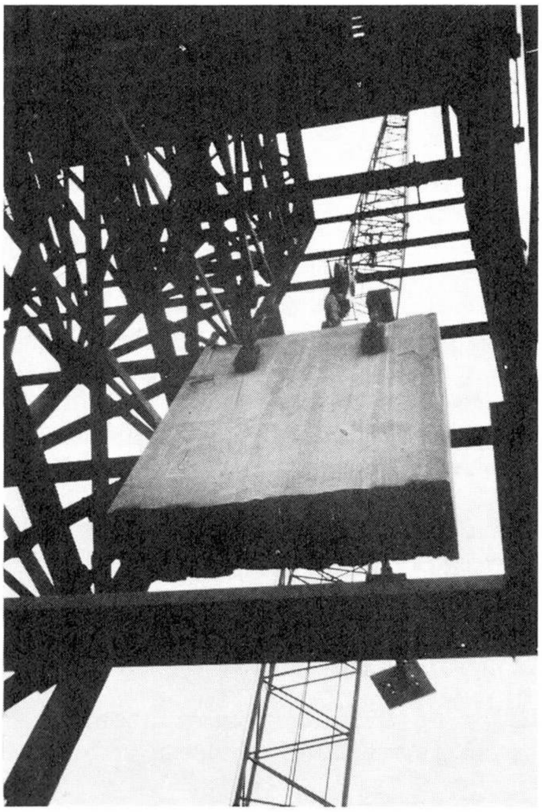

Fig. 9 Column demolition

9. STEEL BEAMS

Steel was selected for the longer 85.4 span primarily because it would be lighter in weight, and therefore pile loading would be less and erection easier than for a concrete span. The boxes were fabricated from high tensile steel (Grade 350 L15) and were fully welded. Plate thicknesses were up to 32 mm and the philosophy adopted, to minimise time and cost, was to use thicker plate sizes and less stiffening. It was necessary to replace the system of balanced rocker and knife edge bearings at each pier, which was done by providing six bearings to match the six concrete beam positions. Therefore the bearing locations generally fall some distance from the steel box centre lines, necessitating stiff steel boxed transverse cross beams at the ends. The "MERRISON" [9] Interim Design Rules were used for the design of the steel boxes and were useful in assessing the results of

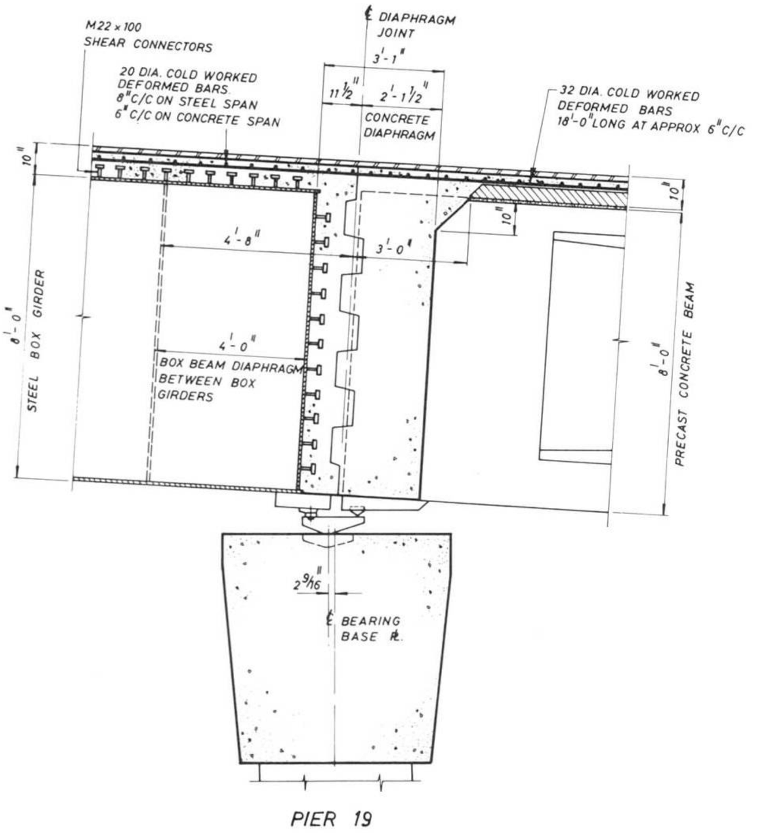

Fig. 10 Continuity connection steel/ concrete spans

the finite element analyses of the end crossbeams. The continuity detail between the steel and concrete spans over the piers (Fig. 10) was similar to that used in the original design.

The depth of the girders was generally 3.6 m tapering to the same depth (2.4 m) at the ends as the adjacent concrete beams. To minimise the appearance problem of the missing pier a light non-structural fascia was attached to the outer boxes, so that the extra depth of the steel beams was in shadow.

10. ERECTION

The erection scheme was dictated by the heavier and larger steel boxes (250 tonnes). Initially, launching was considered but this was ruled out because the support reactions were too heavy for the existing concrete deck. It would also have made the concurrent widening of the bridge impossible.

Fig. 11 Box girder erection

The adopted scheme used luffing 'A' frame lifting derricks specifically designed by the Contractor for the purpose (Fig. 11). By using temporary cantilever extensions at the ends of the boxes, they were lifted outboard of the towers as a single unit and positioned on top of the towers. The boxes were towed to site on a barge and then lifted. The operation from mooring of the barge to lifting the girders 40 m, and positioning them on temporary bearings on the falsework towers took about 20 minutes.

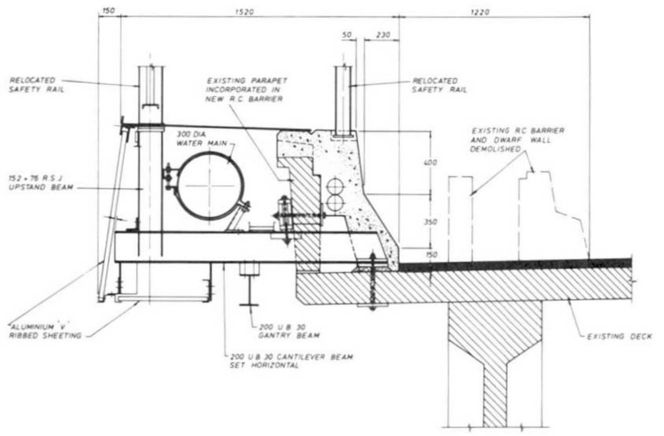

Fig. 12 Bridge widening details

The six precast concrete beams for spans 19-20 were erected in the same manner

11. BRIDGE WIDENING

While the bridge was out of service the opportunity was taken to widen it from four to five lanes over its full length. This permitted the operation of a rather more comfortable 3:2 tidal flow system rather than 3:1 as previously. The widening was achieved by cantilevering new footways and using the existing concrete deck for the five lanes (Fig. 12).

The 42.7 m span, prestressed concrete beams in the main viaducts were constructed segmentally. Due to the extra load from the widening, the outer beams were in tension at working loads and for serviceability reasons it was decided to strengthen them by adding external tendons in a "Vee" shape one on each side of the web (Fig. 13). The tendons were 8 x 12.5 mm strands grouted into a 75 mm steel pipe and were stressed by pulling them down at mid span, and anchored by bolting to the mid span saddle to give an uplift force of 38 tonnes.

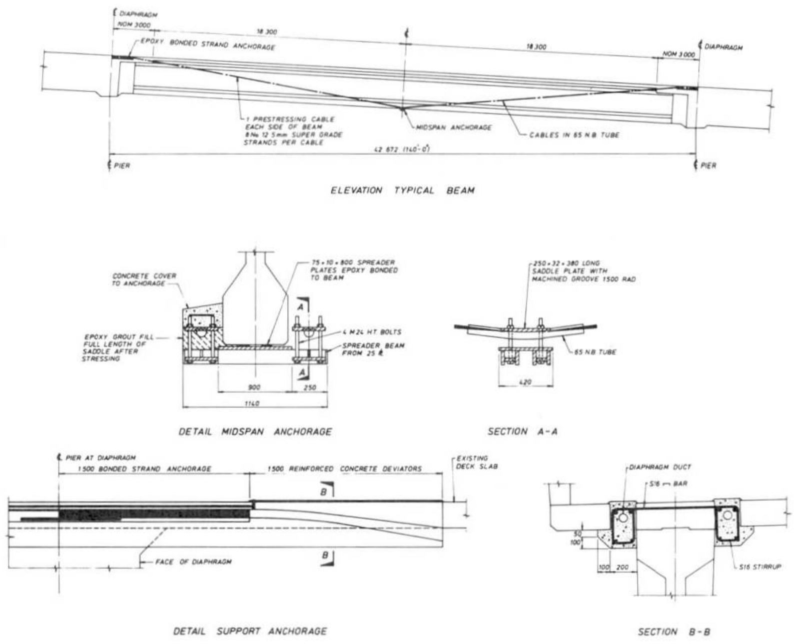

Fig. 13 Beam strengthening details

It was originally intended to anchor the tendons in the deck by breaking out an area of concrete and casting a traditional anchor block. To reduce the required demolition and avoid cutting the main transverse deck reinforcement, the Contractor, Pearson Bridge (Tas.) Pty. Ltd. proposed cutting a slot 3 m x 200 mm (Fig. 14) into which the bare strands were anchored using an epoxy with fillers. The method worked very well.

Fig. 14 Slots

12. CONCLUSION

The bridge was opened to traffic on the 8th October, 1977 ahead of schedule. The restoration consisted basically of demolishing a part of the original bridge and rebuilding "two" bridges, one of steel and the other of concrete. As a result the whole job was on a learning curve with no production runs available. The task involved many unusual engineering features and decisions and a huge number of engineering hours.

The successful completion of the project slightly ahead of time is a tribute to all who were involved.

13. ACKNOWLEDGEMENTS

This paper is published with the kind permission of the Joint Tasman Bridge Restoration Commission and of the Department of Main Roads, Tasmania.

Imperial units have been used on some of the figures in this paper, as all existing information concerning the structure was in these units.

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