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Monitoring of a post-tensioned Bridge during Demolition

Contrôle d'un pont a précontraite postérieure au cours de démolition

Überwachung des Abbruches einer Spannbetonbrücke

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SUMMARY

The deterioration of the precast concrete segments in a 25 year old post-tensioned bridge made it necessary to demolish and replace the superstructure. Debonding of the severed prestressing tendons was generally limited to three metres from the cut point. Consequently, excessive levels of prestress force remained in the deck sections over the intermediate piers as self-weight moments decreased. This action caused yielding of the longitudinal steel at the in-situ joints and non-linear behaviour of the structure. Strain gaugin of prestressing tendons during demolition indicated that long-term losses of prestress in the central span were in the range from 35-50%.

RESUME

La détérioration des segments de béton préfabrique d'un pont à précontrainte postérieure, qui a 25 ans, a rendu nécessaire la démolition et le remplacement de la superstructure. La perte d'adhérence des tendons coupés précontraints était limitée à trois mètres du point d'incision. Par conséquent, des niveau excessifs de la force précontrainte sont restés dans les sections du pont sur les piles intermédiaires comme les moments dus au poids propre diminuaient. Cette action a conduit l'acier longitudinal à céder aux joints de construction, et au comportement non linéaire de la construction. L'indication de la tension des tendons précontraints en cours de démolition à montré, que les pertes de précontrainte à long-terme dans la traveé centrale étaient entre 35-50%.

ZUSAMMENFASSUNG

Die Verschlechterung des Instandes der vorfabrizierten Betonsegmente einer 25-jährigen Spannbetonbrücke erforderte Abbruch und Wiederaufbau des Brückenträgers. Der Verbund der Spannglieder löste sich allgemein nur auf etwa drei Metern von der Einschnittstelle. Daraus resultierten übermässige Vorspannkräfte über den Zwischenstützen, da die Momente infolge Eigengewicht abnahmen. Dies führte zum Fliessen der Längsbewehrung in den Fugen und zu nichtlinearem Verhalten des Brückenüberbaues. Dehnungsmessungen an den Spanngliedern während der Abbrucharbeiten zeigten, dass die Langzeitverluste der Spannkraft im Mittelfeld 35 - 50% betragen haben.

1. INTRODUCTION

The Taf Fawr Bridge, built in 1964, carried the A470 trunk road over a tributary of the River Taff north of Merthyr Tydfil, South Wales. The bridge was approximately 30m above the river and had an overall length of some 144m, in three continuous spans of 39m, 66m, and 39m. Designed as a segmental post-tensioned structure, the deck was assembled as a balanced cantilever. Full details of the original design and construction procedure were described by Lundgren and Hansen [1].

The segments and joints were numbered consecutively from the west end (Fig. 1). Each segment comprised four, 3m long, precast I-sections joined by in-situ top and bottom slabs to form a three cell box structure. The precast I-sections were temporarily held in place during erection by a diagonal Macalloy bar passing through the webs at the joints. The Macalloy bars remained in position in the permanent structure and were fully grouted. Longitudinal post-tensioning was then applied to the precast units so that two 19 wire/25mm diameter strands were anchored at each cantilever end, running through the previously assembled I-sections. The continuously reinforced slabs were cast and further post-tensioning was applied.

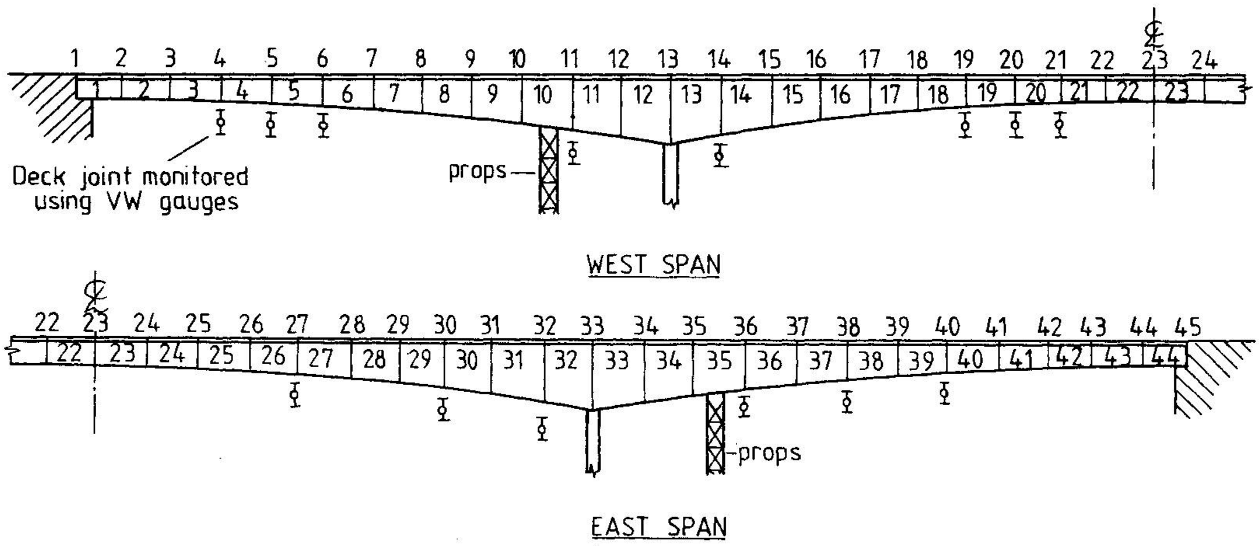

Fig. 1 Deck segment and joint numbering sequence

Tendons were provided in the top of the deck throughout the length of the superstructure to counteract the effects of moments induced during erection and under working conditions. Further tendons were added in the bottom slab, in the centre span and near the abutments, to resist the sagging moments after the bridge was made continuous.

In 1985 the decision was made to demolish the superstructure due to general deterioration of the outer precast sections. This deterioration enabled water and de-icing salts to leak through into the prestressing ducts causing corrosion of some of the tendons. The reinforced concrete piers were still in good condition and it was economically feasible to replace the deck alone. In order to remove the deck with the least damage to the piers, the dismantling process demanded a reversal of the construction sequence. Some operations were not wholly reversible, such as the reduction of prestress in step with the relief of dead load stresses. In addition, the behaviour during demolition was not entirely predictable since the extent of debonding on cutting the tendons was unknown. A number of monitoring procedures were utilised to minimise the uncertainty and to ensure that demolition was carried out in a safe and controlled manner.

The monitoring work was extended to include an assessment of the levels of stress remaining in the superstructure. Long-term losses of prestress, from tests on previous post-tensioned structures, have generally exceeded the original design predictions [2-4] and new techniques are being developed for determining the residual levels of prestress [5].

2. DEMOLITION SEQUENCE

Initially the superstructure was divided into two parts by cutting transversly at midspan. The central box units were then isolated by removing the outer I-sections. After providing temporary props under each side span, the connections between the superstructure and the abutments were severed. The box units were then lowered into the valley, maintaining stability by ensuring excess weight remained on the side span. Work commenced on the west span side sections and progressed onto the east span. The lowering of the west span central units ran concurrently with the removal of the east side sections. Finally, the east span central units were removed using the lifting frames transferred from the west side.

Initial trials, using a diamond saw to cut the midspan position, proved ineffective as the cantilevers dropped trapping the blade. The demolition work proceeded using a combination of pneumatic jack hammers and a Montabert breaker. The breaker was used to isolate each unit prior to removal by crane or lifting frame.

3. INSTRUMENTATION

3.1 Structural monitoring

During the removal of the central units four principal features had to be considered for monitoring, the first two being of prime importance from a safety aspect.

- Cutting poorly grouted cables could lead to excessive debonding of the steel occurring over several segments from the cut position. This could result in either ejection of an anchorage or problems in the demolition process. Only two cables connected some of the units to the remaining cantilever which had to support both the lifting frame and the end unit being demolished. Initial trials were carried out in the top slab of the north edge section of segment 14, the extent of debonding being assessed by monitoring the change in concrete surface strains which occurred along the line of the severed cables. Further debonding checks were carried out in the central units of segments 26 and 27.

- Excessive compressive stresses might develop in the top flange and tensile strains might be induced in the bottom flanges of the units near the piers. This effect would be due to unreleased prestress in the top slab, which could exceed the cantilever moments as self-weight was reduced.

- The pier reactions and rotations had to be monitored in order to ensure that the concrete hinge at the base did not crack as the weight of the deck was removed.

- In conjunction with the monitoring of the pier the general stability of the cantilever had to be assessed as each unit was removed.

In order to detect the first possible form of failure, vibrating wire (VW) strain gauges were attached to the inside of the box units across the in-situ joints at the ends of both the midspan and side span cantilevers. A sudden loss of prestress would be immediately apparent at the joints if tensile stresses began to develop at the top of the web. The second form of failure would produce tensile stresses across the joints in the bottom flange adjacent to the piers. Thus VW gauges were placed in these positions and adjacent to the temporary props. The information gained on the west side resulted in a revised gauging pattern for instrumenting the east span (Fig.1).

All structural monitoring of the superstructure was carried out remotely so that the demolition programme was unaffected and the risks to the monitoring team and the Resident Engineer's staff were reduced. Readings from the strain gauges and temperature sensors were taken from the adjacent temporary Bailey bridge. Changes were monitored on removal of each central unit and after movement of the lifting frames.

The last two monitoring checks could be carried out by instrumenting the concrete piers and steel props. The large area of the piers meant that little strain change would be noted and the readings might be swamped with temperature effects, however, two 140mm VW strain gauges were placed on either side of the pier close to the outer faces. In addition, two 65mm VW gauges were attached to each of the four props, on the neutral axis, to monitor the change in load due to the removal of each central unit. A number of the strain gauges applied also had temperature sensors so that corrections could be made for changes in the weather.

3.2 Research monitoring

Although the monitoring work was primarily confined to the removal of the central box units, checks were carried out on the level of residual prestress in the strands and the concrete stresses in the webs and flanges of the box. Three methods were used to assess the levels of residual stress in the superstructure:

- The prestress remaining in selected tendons was determined from measurements of the change in strain which occurred when a few wires were cut. Short lengths of strand were exposed so that a Demec gauge could be used to measure the released strains. Although complementary laboratory tests are required to interpret the strains measured, this appears to be the most direct and reliable method for determining residual prestress levels.

- An indirect method, involving the instrumentation of concrete cores, taken from the webs and flanges of units 25 and 27, was used to help in the assessment of the in-situ concrete stresses at sections where high shear stresses had caused severe cracking of the webs. Analysis of the data, obtained by taking strain measurements on and around the cores, relied upon a theoretical analysis of the effects of a hole introduced into a stressed infinite plate [6]. As the strains released around and across the cores relate to the residual stresses in the section, this method produces a summation of the effects of the prestress in the strands and the local stresses due to self-weight.

- The demolition of the structure itself provided another means of determining the concrete stresses. By instrumenting a complete unit and recording the total release of strain, it was possible to relate the stresses released at a section to the cantilever moment due to self-weight and residual prestress.

4. RESULTS

4.1 Structural monitoring

4.1.1 Bonding

The north edge section of segment 14 was selected for initial debonding trials and the cables were exposed at either end of the unit so that the strands could be cut in increments. This also provided an opportunity to measure directly the level of prestress in these strands.

Generally the trials showed that fully grouted tendons would not debond more than a fraction of the length of a unit, ensuring that virtually all the residual prestress force would be present at the next in-situ joint. During progressive cutting of a tendon, the changes in strain on the adjacent concrete surface illustrated the debonding characteristics (Fig.2).

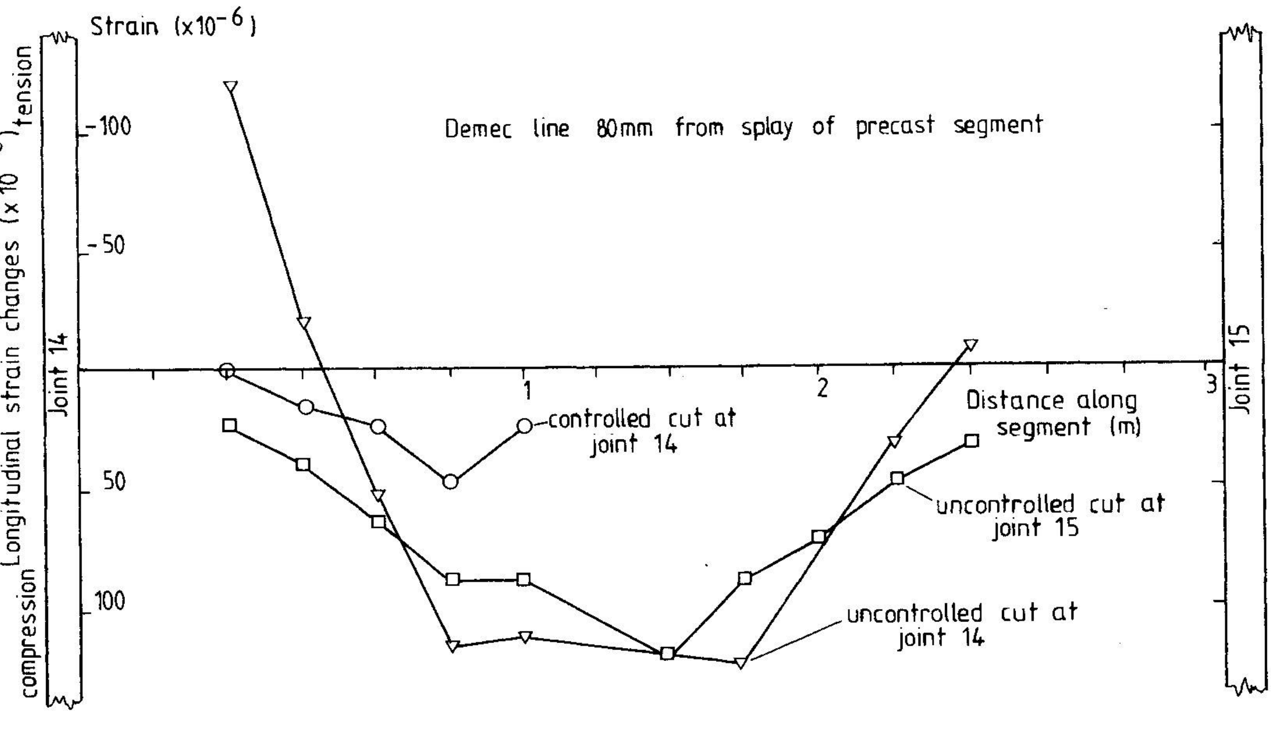

Fig.2 Typical debonding curve.

The curve for the controlled cut at point 1 relates to the cutting of 3 wires in the nearest 19 wire strand at joint 14. The curves for uncontrolled cutting at points 2 and 3 relate to the severing of all the tendons at joints 15 and 14 respectively. As the cutting at point 3 (joint 14) was carried out last, the corresponding curve shows a symmetrical distribution in the strain changes. Each half of the curve is of a form associated with the transfer of stress in the transmission zone of a pretensioned tendon. High strain gradients near the cut end of the tendon are a consequence of the transfer of force from the steel to the concrete by bond. The transmission length is of the order of a metre. In the absence of bond, a fairly uniform tensile strain change would probably be experienced, relating to the release of the general prestress in the segment.

Further checks on the debonding were made in the central units 26 and 27: the results confirmed the original findings. It was concluded that for 60% of the tendons, prestress would be re-established well within the length of a unit. This meant that there was likely to be a substantial increase in the stresses over the piers which could be monitored by the VW strain gauges positioned across the nearest joints.

4.1.2 Behaviour of joints

Generally the strain changes due to the removal of the first box units were relatively small. The readings taken across the joints furthest from the piers were closely predicted by elastic theory. However, as demolition progressed the strain changes in the joints near the piers increased dramatically. This effect was particularly noticeable for joints 14 and 32 on the centre span. Although joint 32 did not initially suffer significant strain changes, after the first five midspan units had been removed the apparent tensile strains increased to become similar to the readings from joint 14 (Fig.3).

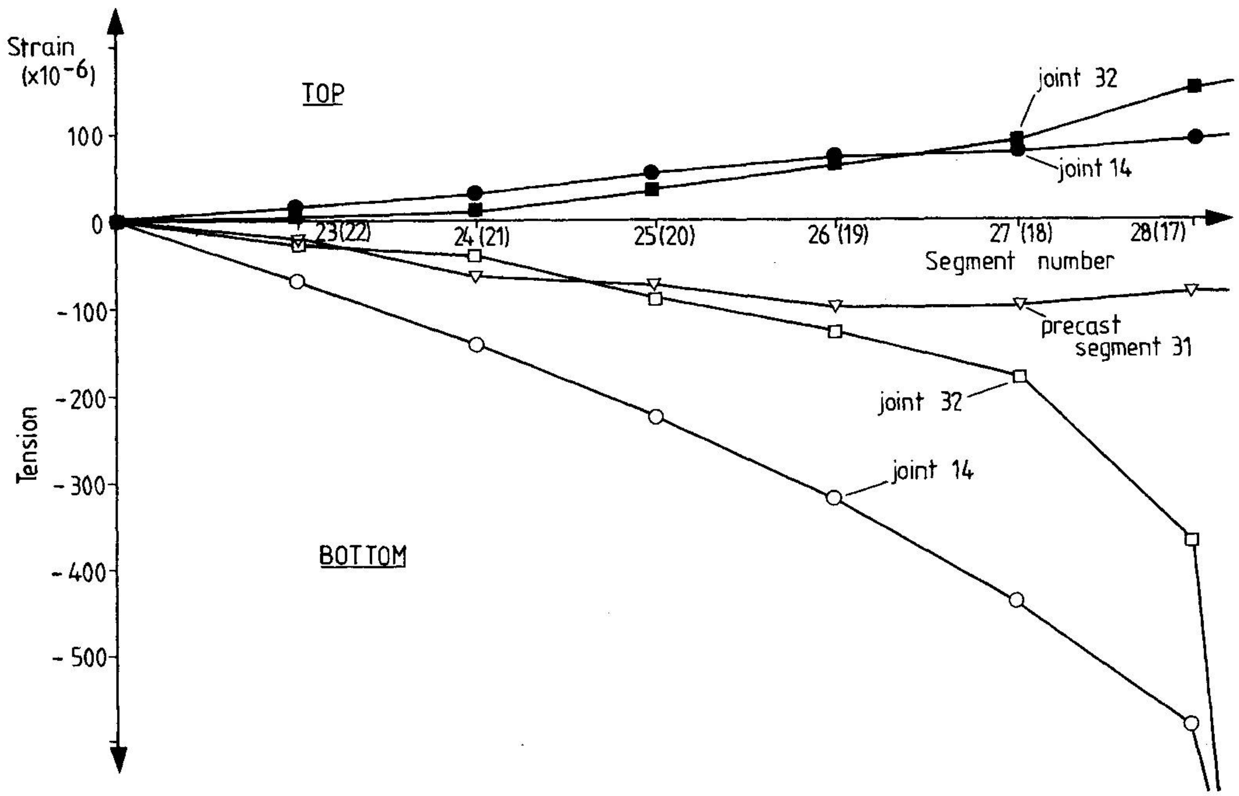

Fig. 3 Comparison of strain changes on south side.

As joint 32 began to show signs of cracking the strain gauge in the adjacent unit No.31 recorded no further increase in strain. The unit could no longer transmit forces below the neutral axis, except by yielding of the mild steel bars in the bottom slab and the two diagonal Macalloy bars that passed between the segments. It appeared that the bottom flange steel would yield well before the demolition sequence was completed and that conventional prestressed concrete theory could not be used to predict the stresses remaining in the cantilever sections as the joints did not retain the full second moment of area associated with the precast units.

4.1.3 Stability checks on piers and props

As the piers were such massive structures, temperature variations were likely to swamp the small strain changes caused by removing the central units. Hence the need to monitor the gauges on the steel props where the strain variations were relatively large. The strain changes were converted into equivalent loads on the falsework and hence the reactions on the piers could be estimated. Constant monitoring of the props showed that no unusual effects occurred during the demolition process.

It was possible to determine the effective Young's modulus for the concrete pier by monitoring the effects, of removing a unit, on the pier and prop. Unit 17 on the midspan side of the pier was chosen and, although the pier strain readings were subjected to the errors caused by the low magnitude, a realistic value of 31.4 kN/mm^2 was obtained for the modulus. A similar exercise was attempted during the removal of unit 39 on the east side span. Unfortunately temperature effects completely swamped the extremely small strain changes in the pier.

4.2 Research monitoring

4.2.1 Direct measurements

Direct measurements of the residual strain in the prestressing strands indicated losses ranging from 35-50% for both units 25 and 26. These results are comparable with measurements taken from other post-tensioned segmental structures of this age [2-4]. The bottom slab cable in unit 25 indicated losses of 35% which showed that debonding losses due to the initial cut some 8m away at midspan had not occurred.

Measurements in segment 14 produced the highest residual steel strains recorded and suggested losses of only 26%. More measurements were needed in the vicinity of the supports to confirm whether the losses were generally lower in this region.

4.2.2 Indirect measurements

The coring method indicated a general pattern of consistency, although there was a degree of unexpected behaviour. In particular, the high shear stresses in the webs caused unusual effects across and around the cored holes because of the high tensile stresses in the shear steel. Only the strain changes on the concrete cores could be considered to give a guide to the stresses in the section. The major principal stresses from the cores in the north web of unit 25 appeared to lie in the range from 2.3-2.7 N/mm^2 at an angle of 58^o to the longitudinal axis. The stresses from the south web were higher at 3.3-4.2 N/mm^2. There was a general degree of consistency in the angle of the major compressive stress of approximately 55^o to the horizontal.

The results from the coring trials carried out through the top slab of unit 27 and the bottom slab of unit 25 showed that these areas were under complex stress conditions. No definite patterns could be ascertained for the various gauging positions as there was very little consistency among the readings. However, in the top slab of unit 27 there were indications from the core results that the longitudinal compressive stresses ranged from 2.0-3.0 N/mm^2. In the bottom slab of unit 25 both core readings indicated longitudinal stresses of 2.5-4.0 N/mm^2.

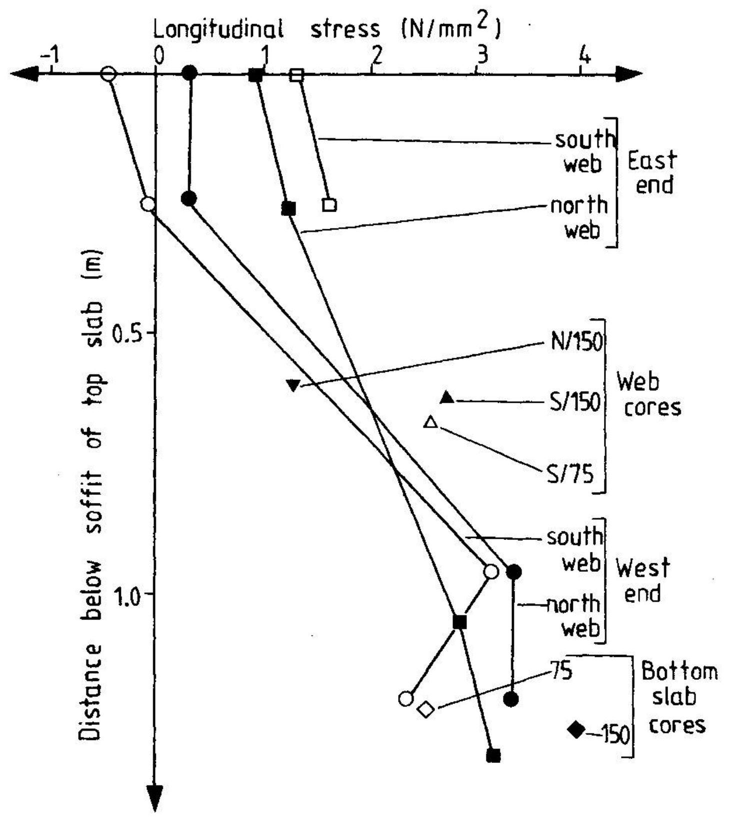

Fig. 4 Longitudinal stresses in unit 25

The stresses derived from the coring method were compared with those obtained from the complete removal of unit 25 (Fig.4). These longitudinal stresses were in reasonable agreement for both the north and south webs at each section although, as expected, there was a distinct difference between the two sections. Despite the fact that the conditions were not identical for the two trials, the results were in good agreement. This indicated that the coring method provided useful indirect evidence on the state of stress in the webs and flanges.

5. CONCLUSIONS

The demolition of Taf Fawr Bridge provided the opportunity of applying various monitoring techniques. The results contributed not only to the development of these methods but also to the safety of many of the operations. It was shown that when a well grouted tendon was severed, the prestress was probably re-established in the concrete within 1m of the cut and certainly well within the 3m length of a unit. For this reason,and because most tendons were found to be well grouted, the units on the central span could be dismantled with an adequate factor of safety.

The total loads on the temporary props, caused by the removal of these units, was closely monitored and corresponded with the reactions anticipated by the contractor. As the demolition progressed and the level of prestress increased in the remaining units near the piers, cracking developed at the joints in the bottom flanges and the strain distribution became non-linear. Warning of the impending yield of the longitudinal reinforcement, in the bottom of the in-situ concrete at the joint, was provided by the VW gauges, but the residual resistance of the section in flexure and shear could not be predicted.

Residual prestrain measured on the tendons indicated likely prestress losses in the range from 35-50%, although in the support region this may have been considerably lower at 26%.

An indirect method using 150mm and 75mm diameter cores can provide a basis for assessing the in-situ stresses in concrete structures with little damage. The results from the coring trials corresponded closely with the total release of stress which occurred on removing unit 25.

6. ACKNOWLEDGEMENTS

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